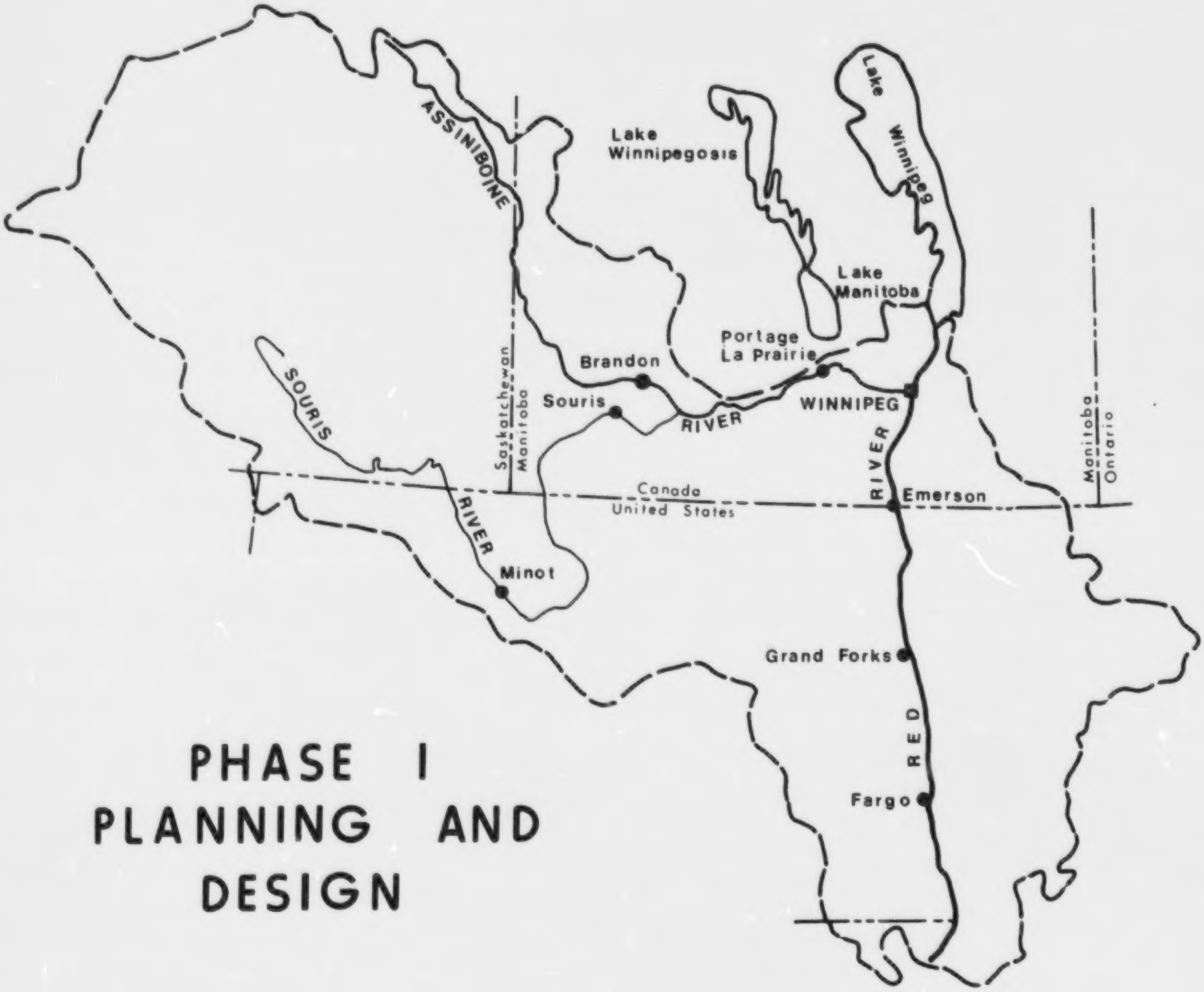
CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
AGREEMENT RESPECTING FLOOD FORECASTING

# MANITOBA RIVER FORECAST DEVELOPMENT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Manitoba
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Canada
United States
RIVER
Emerson
RIVER
Minot
Grand Forks
RED
Fargo

## PHASE I
## PLANNING AND DESIGN

Environment Canada

Manitoba Natural Resources

SEPTEMBER 1985

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AGREEMENT RESPECTING FLOOD FORECASTING

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## PHASE I
## PLANNING AND DESIGN

Environment Canada Manitoba Natural Resources
Winnipeg, Manitoba
September, 1985

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# Synopsis

The primary objective of Phase One of the Canada-Manitoba Flood Forecasting Agreement was to determine if flow forecasts on southern Manitoba streams could be significantly improved at a reasonable cost. Several stream flow simulation models were evaluated against agreed upon criteria. Three promising models of various levels of complexity were chosen for detailed evaluation in a Pilot Project on the Boyne River. Phase One also involved studies of river channel routing models and tests of real-time data acquisition systems for flood forecasting.

The three models selected for detailed testing in the Pilot Project included the complex Hydrologic Simulation Program - Fortran (HSPF), the less complex Streamflow Synthesis and Reservoir Regulation (SSARR) and the simple Single Linear Unit Reservoir Parametric (SLURP). These models were calibrated on the entire Boyne River watershed upstream of Stephenfield (approximately 900 km$^2$ ) and on Roseisle Creek, a 195 km$^2$ tributary of the Boyne River. Three above average spring runoff events were used. Weather and streamflow data during the summers preceding the selected spring runoff events were also used in the calibration process.

The calibrated models were tested using three additional high-flow spring runoff events. Performance scores were calculated for each model and for the statistical unit hydrograph forecasting procedure, MANAPI which is currently used. For the Boyne River watershed upstream of Stephenfield, the HSPF model performed the best. The SSARR model and the MANAPI model had comparable but less accurate scores. The SLURP model was the least accurate. For Roseisle Creek, the HSPF model again performed the best, followed by the SSARR, SLURP and MANAPI models in order of decreasing accuracy.

The HSPF hydrologic simulation model is recommended over the other models tested in the Pilot Project based on its superior performance and its more rigorous representation of the physical runoff processes. Results of a preliminary economic analysis suggested that the benefit/cost ratio of application of the HSPF model to selected tributaries of the Red, Assiniboine and Souris Rivers in Phase Two of this study would be greater than one. On the advice of the Technical Subcommittee, the Steering Committee of the Canada-Manitoba Flood Damage Reduction Program recommended that Phase Two of the study proceed. The governments of Canada and Manitoba approved the required funding. Work on Phase Two began in February, 1984. During Phase Two, the HSPF hydrologic model will be applied to selected tributaries where improved flow forecasts will have the greatest impact on the accuracy of flood forecasts for the Red, Assiniboine and Souris Rivers.

# Résumé

Le principal objectif de la phase un de l'accord Canada-Manitoba concernant la prévision des inondations était de déterminer si l'on pouvai't apporter des améliorations importantes et à prix raisonnable au système de prévisions des écoulements des rivières du sud du Manitoba. Plusieurs modèles de simulation de débits furent évalués en fonction de critères établis à l'avance. Trois modèles prometteurs, de complexités variées, ont été retenus pour faire l'object d'une évaluation plus poussée dans le cadre du projet-pilote réalisé sur la rivière Boyne. La phase un prévoyait également une étude de modèles de calcul du cheminement d'une intumescence d'un cours d'eau et des systèmes d'enregistrement des données en temps réel pour la prévision des inondations devaient être testés.

Trois modèles ont été retenus afin d'être testés plus en détail au cours du projet-pilote; il s'agit, par ordre décroissant de complexité, du programme de simulation hydrologique Fortran (HSPF), de la synthèse du débit et de la régulation du réservoir (SSARR) et du modèle paramétrique de réservoir linéaire unique (SLURP). Ces modèles ont été étalonnés sur tout le bassin hydrographique de la rivière Boyne en amont de Stephenfield, (superficie de 900 km$^2$ ) et sur celui du ruisseau Roseisle, un affluent de la rivière Boyne, qui s'étend sur 195 km$^2$ . On utilisa pour cela trois crues printanières. On se référera également au cours de l'étalonnage, aux données météorologiques et aux données de débits relévees au cours des étés qui ont précédé les occurrences de crues printanières.

Les modèles étalonnés furent testés au cours de trois autres occurrences de crues printanières. Pour chaque modèle, on calcula les résultats obtenus, ainsi que pour la méthode de prévision hydrographique par unité statistique, MANAPI, qui est actuellement utilisée. Pour le bassin hydrographique de la rivière Boyne en amont de Stephenfield, c'est le modèle HSPF qui s'est avéré être le meilleur tandis que le modèle SSARR et le modèle MANAPI ont donné des résultats comparables mais moins précis. Quant au modèle HSPF s'est une fois de plus avéré être le meilleur suivi par ordre décroissant de précision des modèles SSARR, SLURP et MANAPI.

On recommande donc le modèle de simulation hydrologique HSPF plutôt que les autres modèles testés au cours du projet-pilote parce qu'il a donnée de meilleurs résultats et une description plus exacte des écoulements réels des eaux. Une analyse économique préliminaire suggère que le rapport coûts/bénéfices de l'application du modèle HSPF à certains affluents choisis des rivières Rouge, Assiniboine et Souris au cours de l'exécution de la phase deux de cette étude serait supérieur à un. Sur les recommandations du sous-comité technique, le comité de direction du programme Canada-Manitoba de réduction des dommages due aux inondations a préconisé que l'on passe à la phase deux de l'étude. Les gouvernements du Canada et du Manitoba ont approuvé les fonds nécessaires et les travaux ont commencé en février 1984. Au cours de cette phase, le modèle hydrologique HSPF sera appliqué aux affluents pour lesquels l'amélioration des systèmes de prévision de l'écoulement réel augmentera la précision des prévisions des inondations des rivières Rouge, Assiniboine et Souris.

# Acknowledgements

This study was conducted under the direction of the Technical Subcommittee on Flood Forecasting. Committee members were:

Environment Canada:
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G. B. Atkinson replaced by R. L. Raddatz
J. G. Way replaced by G. R. McCulloch (Co-Secretary)

Manitoba Natural Resources:
V. M. Austford (Co-Chairman)
R. J. Bowering
A. A. Warkentin (Co-Secretary)

A study leader was provided by Manitoba Water Resources to supervise the study under the direction of the Sub-Committee. Study staff were appointed under the terms of the Agreement. The study group were:

| | |
|---|---|
| Study Leader | A. A. Warkentin |
| Flow Forecaster | D. E. Kelln |
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Drafting support was provided by J. Mamott and C. Wojciechowski. Computer support was provided by H. Sager, G. Morgan and D. Walker of MWRB and A. Glennie of CWRB.

The draft of the report was written by the Study Leader with the help of notes made by the Study Staff. The Technical Sub-committee reviewed the various drafts and made revisions as necessary. The drafts were typed by P. Kornago of MWRB and the manuscript was typed by P. Welwood of AES. T. Gallagher copied and assembled the reports.

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# List of Acronyms
(Excluding Runoff Model Parameters)

AES - Atmospheric Environment Service of Canada
CORPS - Army Corps of Engineers (U.S.A.)
CWRB - Canada Water Resources Branch
DATS - Data Acquisition and Telemetry System
DCP - Data Collection Platform
EPA - Environmental Protection Agency (U.S.A.)
ET - Evapotranspiration
HEC-1 - Hydrologic Engineering Centre-Flood Hydrograph Package (U.S.A.)
HSPF - Hydrologic Simulation Program Fortran
HYMO - Hydrologic Modelling, Problem-Oriented Programming Language
ILLUDAS - Illinois Urban/Rural Drainage Area Simulation
MANAPI - Manitoba Antecedent Precipitation Index Model
MANTHORN - Manitoba Thornwaite Runoff Model
MITCAT - Massachusetts Institute of Technology Catchment Model
MMDW - Minnesota Model for Depressional Watersheds
MWRB - Manitoba Water Resources Branch
NWSRFS - National Weather Service River Forecast System
One-D - One Dimensional Hydrodynamic Routing Model
P - Precipitation
PARAMETRIC - Parametric Runoff Model
PET - Potential Evapotranspiration
PFRA - Prairie Farm Rehabilitatin Administration
QUFM - Queens University Forecast Model
RO - Runoff
RROUT - Runoff and Routing Model
SASK6 - Saskatchewan River (Modified UBC) Model Using 6-layer Hydrologic Soil Moisture Budget
SLURP - Single Linear Unit Reservoir Parametric Model
SSARR - Streamflow Synthesis and Reservoir Regulatin Model
SSARR-4 - Streamflow Synthesis and Reservoir Regulation-Version 4
STORM - Storage, Treatment, Overflow Runoff Model
UBC - University of British Columbia Runoff Model
USDAHL - USDA Hydrographic Laboratory Model
WSC - Water Survey of Canada

## Chapter 1

# INTRODUCTION

The history of flood forecasting in Manitoba dates back more than three decades. After the disastrous spring flood of 1950 on the Red River, the Water Resources Division of the then Department of Resources and Development conducted a study to determine means of reducing the flood hazard of the Red River with respect to the City of Winnipeg. The 1953 "Report on Investigations into Measures for Reduction of the Flood Hazard in the Greater Winnipeg Area" (Water Resources, 1953) involved the study of various proposed flood control works. Three major control works, namely the Red River Floodway, the Portage Diversion and the Shellmouth Reservoir were constructed during the 1960's to reduce the flood threat in Winnipeg. The investigation also included the development of a graphical procedure which could be used to prepare runoff and streamflow outlooks for the Red River at the International Boundary near Emerson. Preliminary forecasts for the Red River at Winnipeg were obtained using either a peak-stage relationship between Emerson and Winnipeg or using a streamflow routing procedure to route recorded and projected tributary flows from Emerson to Winnipeg. These procedures were in use from the mid 1950's up to the mid 1970's.

On the Assiniboine River, graphical runoff procedures for five major sub-basins were developed in a 1952 study by the Prairie Farm Rehabilitation Branch of the Department of Agriculture, Canada entitled "Conservation and Flood Control, Assiniboine River" (PFRA, 1952). One of the five sub-basins was the Souris River. A variation of the unit hydrograph approach was used to estimate the time distribution of runoff for each sub-basin. River routing was then used to produce forecasts at downstream points. In 1955, this forecasting procedure was extended by the Water Resources Branch, Department of Mines and Natural Resources, Manitoba, through the addition of storage routing procedures for six reaches of the Assiniboine River from Kamsack to Headingley. However, due to the laborious nature of doing flow routings by hand, peak discharge relationships were generally used to estimate downstream peaks on the Assiniboine River. These forecasting procedures for the Assiniboine River were in use from the mid 1950's to the early 1970's.

A Flood Forecasting Committee was formed in 1954 composed of senior officials of the Manitoba Department of Mines and Natural Resources, (now Manitoba Natural Resources, Water Resources Branch) the Canada Department of Northern Affairs and National Resources, (now Environment Canada, Water Resources Branch) the Canada Department of Transport, (now Environment Canada, Atmospheric Environment Service) the Greater Winnipeg Water District (now Water Works and Waste Disposal) and the Manitoba Department of Public Works (combined with Manitoba Water Resources Branch since 1959) to scrutinize and approve forecast information derived by the Manitoba Water Resources Branch before release to the public.

The flood forecasting procedures developed during the early 1950's performed reasonably well but were in need of updating by the late 1960's to take advantage of the additional hydrometric data that had been recorded during this interval. Computerization presented the opportunity for more extensive analysis of data and the testing of new methodologies. In 1970 the Water Control and Conservation Branch, Manitoba Department of Mines and Natural Resources embarked on a program of accelerated flood forecasting development. A permanent staff position was established to develop new flow forecasting methodologies. Term staff and summer students were hired to assist. Because hydrologic simulation models were still in their infancy and because funds were limited, the new flow forecasting approaches developed during the 1970's still employed graphical and statistical procedures as well as unit hydrographs. Nevertheless, a significant improvement in forecast accuracy was achieved for the Red, Assiniboine and Souris Rivers through the use of improved techniques of estimating soil moisture, effective precipitation and melt rates, and through the improved analysis of much greater amounts of data by computer. The new methodologies became known as the Manitoba Antecedent Precipitation Index (MANAPI) model. This model is described in Appendix B.

The MANAPI flow forecasting procedure was calibrated on 45 tributaries of the Red, Assiniboine and Souris River watersheds between 1970 and 1977. Computerized Muskingum flood routing procedures were developed for within-bank and for overbank flow conditions for the Red River from Emerson to Winnipeg, for the Assiniboine River from Kamsack, Saskatchewan to Winnipeg and for the Souris River from Minot, North Dakota to Wawanesa. Computer programs were prepared to facilitate the rapid preparation of flow forecasts for snowmelt events and/or any combination of recorded or anticipated rainstorms. These flow forecasting procedures were in various stages of development and use from 1974 to 1977 and in full use from 1978 onward.

The Manitoba Water Resources Branch prepares runoff outlooks during the third week of both February and March of each year. Additional outlooks are prepared when spring runoff is delayed by cool weather, particularly if there is significant additional precipitation. Runoff outlooks are based on existing basin conditions combined with various future weather conditions, usually the median and the upper decile probabilities. Forecasts are prepared daily when spring runoff is actively underway. These operational forecasts are based on recorded data up to the forecast date combined with various specified sequences of future precipitation events.

The numerous floods during the 1970's, many of which were aggravated by severe spring weather conditions, severely tested the updated flow forecasting procedures. In 1974, significant spring flooding occurred on the Red River, along with major flooding on its Manitoba tributaries. This flood was aggravated by a heavy rainstorm which occurred at the end of the snowmelt period. Minor flooding took place on the Red River in 1975 and 1978. In 1979, a major flood of similar magnitude to that of 1950 occurred on the Red River in April and early May. Only minor flooding had been predicted in the March 22, 1979 outlook. This major flood was largely caused by extreme spring weather conditions which had only a 3% probability of occurrence for the period from March 22 to April 30. On the Assiniboine River and the Souris River, the largest flood on record occurred in 1976. Peak discharges were about twice the previous recorded maxima on the Souris River. While record flooding had been predicted, this flood turned out to be much greater than expected. The 1976 flood was not due to severe spring weather but resulted from a very high soil moisture at freeze-up and a much above average winter snowpack. The Assiniboine River and the Souris River also experienced significant flooding in the spring of 1974, 1975 and 1979. Flood damage to homes, farms and small businesses in Canada for the 1979 Red River Flood alone when combined with emergency response costs were estimated to be $26,700,000.

The high incidence of flooding during the 1970's, and the inaccuracy of many of the outlooks and some of the operational forecasts raised some questions as to the suitability of the multiple regression unit-hydrograph approach used for runoff forecasting. While these procedures generally produced quite accurate forecast with inputs within the range of experience they did not perform well in situations such as occurred on the Assiniboine and Souris Rivers in 1976 when the causal parameters were well beyond the limits of experience. Furthermore, a lumped index model such as MANAPI is not suitable in situations where the snowmelt is very irregular in time, particularly if runoff has begun and the remaining snowpack is re-frozen for several days due to a return to cold weather. These factors, together with the ready availability of runoff simulation models in the 1980's made it desirable to determine whether greater forecast accuracy and greater lead time could be achieved by the implementation of a model based on physical representations of the hydrologic processes rather than indices.

Because the Red, Assiniboine and Souris River watersheds are very flat and large, flood flows are retarded in depressions and river channels. This factor makes flood routing a very important component of operational flow forecasting on these rivers. Forecasts with a lead time of seven to 14 days to the peak are possible based primarily on flood routing. There is generally a time lag of seven to 14 days from the time most tributaries have peaked to the time of the peak flow at the major urban centres on the main stem of the Red, Assiniboine and Souris Rivers in Manitoba. During this interval therefore the accuracy of forecasts on the main stem is primarily dependent on the accuracy of "recorded" flows up to the forecast date and on the effectiveness of channel routing of recorded and projected flows. Accurate forecasting of tributary flow is generally the priority when tributary flows are rising rapidly due to snowmelt or due to subsequent rainstorms. The advantage of improved flow forecasting using simulation models would therefore lie mainly in more accurate tributary forecasts. This would result in more accurate flow and peak stage forecasts on the main stems about two to four days sooner than those based only or primarily on recorded tributary flows and flood routings. It should also result in more accurate pre-melt outlooks of spring peak stages on the main stems.

The Agreement Respecting Flood Forecasting, a subagreement to the Canada-Manitoba General Agreement Respecting Flood Damage Reduction, was signed by Canada and Manitoba on March 31, 1981. The Agreement provides for a five year study to develop and implement improved spring flood forecasting procedures for the Red, Assiniboine and Souris River basins in Manitoba. The main objective of Phase One was to determine whether the use of continuous deterministic or parametric hydrologic simulation models could significantly improve the accuracy and lead time of streamflow forecasts over that attainable with the traditional approach based on multiple regression, unit hydrograph and index values. A copy of the Canada-Manitoba Agreement Respecting Flood Forecasting is contained in Appendix H.

The $600,000 study was divided into two phases. Phase One had a maximum limit of $350,000 and a termination date of March 31, 1984. In this phase, a Pilot Project on the Boyne River watershed was used to compare and test three hydrologic models; the simple parametric SLURP model, the intermediately complex and well known SSARR model and the complex deterministic HSPF model. The presently used MANAPI regression model was used as a benchmark. Other Phase One activities carried out in concurrence with the Pilot Project consisted of channel routing studies within the Boyne River watershed and the testing of real-time data acquisition systems. In Phase Two, the forecasting system recommended as a result of Phase One studies was be applied to the Red, Assiniboine and Souris River watersheds in Manitoba to the extent possible within the total allowable expenditure of $600,000 and prior to the termination date of March 31, 1986.

Two hydrologists and one technologist were hired under the Agreement. The study staff were located in the offices of the Manitoba Water Resources Branch. That agency provided a Study Leader and provided overall management of the Study under the direction of the Technical Sub-Committee on Flood Forecasting. Additional direct technical input was provided by agencies represented on the Sub-Committee: The Manitoba Water Resources Branch (MWRB), the Canada Water Resources Branch (CWRB), and the Atmospheric Environment Service (AES).

A brief report entitled, "Summary Report, Phase One, Planning and Design" was prepared in December 1983. This led to a decision by the parties to the Agreement to proceed with Phase Two.

The complete technical report contained herein describes in detail the activities conducted during Phase One of the Flood Forecasting Agreement. While the major portion of the report deals with the Pilot Project, the remaining portions of Phase One studies including channel routing and real-time data acquisition activities are also described.

## Chapter 2

# SELECTION OF HYDROLOGIC MODELS

Eighteen North American runoff simulation models were briefly reviewed in order to ascertain which might be suitable for southern Manitoba conditions. These models are shown in Table 1. The list was limited to those models for which information was readily available. Model selection criteria were then established to serve as an aid in assessing the relative suitability of the models. Major criteria were that the models be deterministic, i.e. as physically-based as possible, and that they be relatively easy and inexpensive to apply. Demonstrated accuracy and proven operational use were desirable but not essential. Table 2 shows the model selection criteria.

Of the 18 models examined, eight were subjected to a detailed qualitative evaluation. The evaluation matrix for these eight is shown in Appendix A. Of these eight, three were selected for detailed quantitative examination. These were the National Weather Service River Forecast System (NWSRFS), the Hydrologic Simulation Program - Fortran (HSPF) and the Single Linear Unit Reservoir Parametric (SLURP). After some initial studies on these models the NWSRFS model was dropped because complete documentation and user manuals were not available. In its place, the well-known Streamflow Synthesis and Reservoir Regulation (SSARR) model was chosen. This left three models of different levels of complexity to be evaluated under the Pilot Project: the complex deterministic HSPF model, the less complex and partially parametric SSARR model, and the simple parametric SLURP model.

The selection criterion relating to the use of deterministic models was thus relaxed somewhat to enable the testing of models of various complexity, cost of calibration and use and data requirements. This gave the study the added dimension of determining the most suitable model in terms of cost as well as performance.

The HSPF model was prepared by Hydrocomp Inc. for the United States Environmental Protection Agency (EPA). It combines the basic hydrologic algorithms of the Stanford IV watershed model developed by Crawford and Linsley in 1966. The basic energy snowmelt equations were originally developed by the North Pacific Division of the United States Corps. of Engineers (CORPS) in the 1950's (Snow Hydrology, 1956). These equations were later updated by E.A. Anderson and N.H. Crawford in the 1960's. HSPF is a complex model in which physical watershed processes are generally represented in a mathematical conceptual fashion.

The SSARR model is a widely used model originally developed for the planning and management of the Columbia River System by the United States Corps of Engineers, North Pacific Division (Snow Hydrology, 1956). The energy budget snowmelt equations are essentially those adopted by EPA for the HSPF model. Unlike the HSPF model, the SSARR model does not compute the negative heat storage of the snowpack nor the albedo of the snow. The lands phase (movement of input moisture on and through the soil, etc.) of the SSARR model is largely parametric and therefore it is not as physically-based as that of the HSPF model.

The SLURP model is a simple parametric model developed by the Water Resources Branch, Environment Canada (Kite, 1975). It operates on a daily time step and requires only daily values of temperature, precipitation and potential evapotranspiration as input. The SLURP model has not been used for flood forecasting.

The MANAPI model is a statistical index model developed by the Manitoba Water Resources Branch in the 1970's (Warkentin, 1973). It employs multiple regression based on a product-power type of function and Fletcher Optimization. Unit hydrographs are used to convert runoff to streamflow.

Details regarding the selection of hydrologic models for testing in the Pilot Project are contained in Appendix A.

The HSPF, SSARR, SLURP and MANAPI models are described in detail in Appendix B, including the procedures which these models employ with respect to snowmelt simulation, runoff simulation and the conveyance of simulated runoff.

# Chapter 3

# THE BOYNE RIVER WATERSHED

The Boyne River Watershed was selected for the testing of the streamflow simulation models in the Pilot Project. This watershed is located in southern Manitoba approximately 90 km southwest of the City of Winnipeg, as shown on Figure 1. The watershed was selected because it is typical of tributaries to the Red, Assiniboine and Souris Rivers and, therefore, procedures found to be applicable to the Boyne River Watershed should be applicable to other tributaries. It has a history of flooding and has a reasonable streamflow and meteorological database for model calibration and verification.

The Boyne River flows eastward off the Manitoba Escarpment, through the town of Carman and empties into the Morris River which joins the Red River at Morris, Manitoba. The study concentrated on the 900 km$^2$ drainage area of the Boyne River upstream of the hydrometric station near Stephenfield, shown on Figure 2. Elevations of this watershed range from 282 metres above sea level near Stephenfield to 525 metres in the headwaters. Most of the watershed is under cultivation but there are numerous wooded areas and areas of natural pasture on the escarpment. Soils consist of fairly heavy clay loams on the escarpment and upland areas with light, sandy materials in the sub-escarpment.

A major tributary of the Boyne River, known as Roseisle Creek, joins the Boyne River just upstream of the Stephenfield Reservoir at the downstream end of the study area. Its drainage area is 195 km$^2$ . Streamflow simulation models were also tested separately on this tributary and compared to the tests in the complete watershed.

Detailed information on the Boyne River watershed is contained in Appendix C.

# Chapter 4

# PILOT PROJECT STUDY FRAMEWORK

The primary objective of the flow simulations on the Boyne River watershed was to compare the performance and suitability of the selected models, i.e. the HSPF, SSARR, and SLURP models. A sub-objective was to determine whether it was necessary to subdivide the watershed into segments with unique characteristics with respect to topography, soil type, vegetation, drainage and climatology, or whether these diverse regional characteristics could be averaged for the entire watershed without seriously affecting model performance. These tests were only attempted using the HSPF and SSARR models.

It was decided that simulated model flows would be compared at two locations, the Boyne River at the Water Survey of Canada (WSC) streamflow gauge near Stephenfield (just downstream of the Stephenfield Reservoir), and Roseisle Creek at the WSC hydrometric station on this stream. These stations are shown on Figure 2. The location near Stephenfield was chosen instead of the WSC gauge located further downstream near Carman because the quality of the recorded data appeared to be better near Stephenfield, particularly for large floods when overflows occur near the Carman gauge.

Roseisle Creek was chosen as a second basin for model comparisons for two reasons. Firstly, Roseisle Creek is much smaller than the entire Boyne basin; it would provide some information on how basin size would affect model performance. Secondly, simulations on Roseisle Creek would present the opportunity to compare model performance without the complications and potential errors associated with channel routing. With the HSPF and SSARR models, channel routing of flows was undertaken for the watershed upstream of Stephenfield.

Since the emphasis of this study was to develop improved spring flow forecasting procedures for high flows, the available hydrometric records on the Boyne River were examined to determine what spring runoff events would be consistent with the goal. Six above average spring runoff events had occurred in recent times. Major floods took place in 1974 and 1979 and a flood of intermediate size flood took place in 1970.

Spring flows were bank-full in 1969, within-bank but well above average in 1971, and just above average in 1976. Based on considerations of runoff conditions, factors causing the runoff, and the availability of meteorological data, the models were calibrated on spring data for 1970, 1976 and 1979 and were verified on spring data for 1969, 1971 and 1974.

In order to compute fall (freeze-up) watershed conditions preceding a given spring calibration year, the models were first run on data for the preceding summer and fall. It was assumed that the computed basin soil moisture storages at freeze-up would remain relatively unchanged during the winter due to frozen ground conditions. The soil moisture conditions at freeze-up were thus used directly as starting conditions for the spring calibrations. For the HSPF model, summer runs were also used to calibrate model process parameters.

The snow accumulation and ablation components of the models were not run. Winter losses from the snowpack are typically very small in Manitoba due to the consistently cold temperatures which usually occur. Occasionally, mild winters occur and do produce a significant loss in snowcover, but historically there has been no spring flood threat following such winters. The decision not to model snowmelt accumulation and ablation was therefore made largely because the benefit of doing so would likely be small compared to the associated time and cost of data preparation and modelling.

Cumulative winter precipitation obtained from isohyetal analyses was used to estimate snowcover prior to break-up. Sublimation losses, based on the Meyer Formula (Meyer, A.F. 1942) for lake evaporation, were subtracted from the cumulative winter precipitation. While this formula was not designed to compute sublimation losses from snowcover, it appeared to produce reasonable loss estimates based on past observations of winter temperatures and snowpack depletion.

The HSPF and SSARR models were run on a daily time interval for the antecedent summer/fall simulations. This time interval for the antecedent simulations appeared to be adequate for the purpose of computing basin moisture storages at freeze-up. However, with the HSPF model it was found necessary to use hourly data to determine some of the process parameters. For the spring simulations the time period used was six hours rather than a day. This was done mainly to increase the accuracy of snowmelt calculations. Snowmelt was found to be quite variable during a day due to diurnal temperature variations, varying cloud cover, rainfall, and frontal passages. It should be noted that the SLURP model operates only with daily data.

The channel routing procedures of the HSPF and SSARR models were applied in a way which minimized the variability associated with different routing procedures, thereby eliminating this aspect from the comparison. SSARR channel routing parameters were adjusted until they produced the same effect as the HSPF routing. The SLURP model does not employ channel routing.

# Chapter 5

# ESTABLISHING THE DATA BASE

The primary source of meteorological and streamflow data for the Pilot Project was Environment Canada. Magnetic tape files of meteorological data were obtained from the Atmospheric Environment Service (AES). Magnetic tape files of hydrometric data were obtained from the Canada Water Resources Branch. Locations of meteorological and streamflow data stations used in this study are contained in Appendix C.

The following meteorological data files were prepared for the spring runoff periods of the three calibration years and the three verification years for daily and/or six-hour time intervals as indicated:

| | |
|---|---|
| Ambient Temperature | (daily and six-hourly means) |
| Dew Point Temperature | (daily and six-hourly means) |
| Wind Speed | (six-hourly means) |
| Global Solar Radiation | (six-hourly means) |
| Precipitation | (daily and six-hourly amounts) |
| Pan Evaporation (May only) | (daily amounts) |

The following data files were prepared for the summer simulations of the condition antecedent to the spring calibration and verification periods:

| | |
|---|---|
| Ambient Temperature | (daily means) |
| Precipitation | (daily amounts) |
| Precipitation - for HSPF Model | (hourly amounts) |
| Pan Evaporation | (daily amounts) |

Files of these meteorological parameters were prepared for five segments of the Boyne River watershed upstream of Stephenfield as well as for the watershed as a whole. The delineation of segments is shown on Figure 1 of Appendix E. Preparation of the files involved quality control of the AES data for specific stations, gap filling of missing records and extrapolation of point source data to create appropriate areal data for watersheds. Many of the parameters such as radiation, wind, rainfall rate, evaporation, and dew point temperature were not recorded within the watershed. It was therefore necessary to develop transfer techniques to transpose data from stations such as Portage La Prairie and Winnipeg to the watershed.

Magnetic tape files of daily streamflow and water level data were used directly for model calibration and verification. Hourly stages and water levels for use in channel routing studies were obtained from the Canada Water Resources Branch.

Winter precipitation data were obtained from isohyetal analyses of total winter precipitation at AES stations, prepared by the Manitoba Water Resources Branch (MWRB). Snowcourse data obtained by AES and MWRB were used to some extent in calibrating the snowmelt component of the models. However, snowcourse data was not used in estimating the water content of the snowpack before spring breakup due to the sparseness of the snowcourse network.

Daily pan evaporation data, adjusted by a pan factor of 0.7 (Kohler et al, 1955), was used to estimate potential evapotranspiration (PE). The resultant evaporations were then multiplied by monthly factors (Blaney and Criddle, 1950) to account for the effect of the typical seasonal state of vegetation growth on PE. With the HSPF model, the latter factors were determined within the model.

Further details concerning these computations are contained in Appendix D which is a detailed documentation of establishing the data base.

# Chapter 6

# CALIBRATION OF HYDROLOGIC MODELS

The HSPF, SSARR, and SLURP hydrologic models were calibrated on the Boyne River watershed upstream of the WSC hydrometric gauge near Stephenfield and on Roseisle Creek. Calibration is the process of adapting generalized hydrologic models to specific watershed conditions.

All models were run on summer/fall data for 1969, 1975 and 1978 in order to determine starting basin moisture conditions for the spring calibrations. For the SSARR and SLURP models the summer/fall simulations involved only rough calibrations using daily input data, to obtain a fair simulation of runoff volumes. Timing of the summer/fall simulated runoff was not considered to be important since the main aim was to compute the moisture storages at freeze-up. For the HSPF model, accurate simulation of summer/fall streamflow was required in order to determine many model process parameters which were not determinable from calibrations on spring runoff. It was necessary to use hourly rainfall inputs in order to properly calibrate this model.

Calibration of the models on spring data involved firstly the calibration of the components of the models dealing with snowmelt. Energy budget procedures were used for the HSPF and SSARR models and a melt factor approach was used for the SLURP model. There was little dependable data upon which to base the snowmelt calibrations; therefore information on the start of runoff and the dates of disappearance of the snowpack was used as best possible. The lands phase of the models was then calibrated together with the timing of the runoff to a stream channel. Finally, channel routing of flows on the Boyne River was employed with the HSPF and SSARR models for calibration of the entire watershed upstream of Stephenfield. Calibration of the SLURP model did not involve channel routing of flows.

The HSPF and SSARR models were calibrated using both a single basin and a segmented basin approach.

The snowmelt calibration data for 1976 turned out to be of little use due to difficulties in estimating the snowpack. The winter of 1975-76 appears to have had abnormally high winter losses from the snowpack which were not apparent until detailed scrutiny had been made. As stated previously, significant winter losses from the snowpack are unusual in Manitoba. Since the 1976 spring streamflows were well within-bank and much lower than in the other calibration years, the limited usefulness of the 1976 spring data did not greatly impede calibration of the models for flood flows.

It was difficult to calibrate the models so as to produce good flow simulations for both 1970 and 1979. It was surmised that there were unusual conditions such as stream channels severely clogged by ice and snow in 1979. Therefore, data from 1970 were relied on more heavily for model calibration.

Calibration results suggested that the segmented basin approach did not produce a better calibration than the single basin approach. Also, calibration difficulties and results for Roseisle Creek were very similar to those for the entire watershed.

Calibration of the models and the calibration results are discussed in detail in Appendix E. Hydrographs showing simulated and recorded flows for the spring calibrations are also shown.

# Chapter 7

# VERIFICATION OF HYDROLOGIC MODELS

The HSPF, SSARR, SLURP and MANAPI models were verified on the spring runoff events of 1969, 1971 and 1974. Verification is the process of testing how well models will simulate streamflow with the model parameters obtained from calibration.

During the spring of 1969, bankfull flows resulted from the rapid melt of an above average snowpack on top of a basin still wet from heavy rains which had occurred the previous fall. Spring flows in 1971 were within-bank but above average and were caused by the rapid melt of an above average snowpack on top of dry soil. A major flood occurred on the Boyne River in 1974, similar in magnitude to the flood of 1979. The 1974 flood was caused by a rapid melt of a 200 percent of average winter snowfall and was aggravated by a 40 millimetre rainstorm at the end of the snowmelt period.

The models were run with meteorological input data for 1969, 1971 and 1974 on the Boyne River upstream of Stephenfield for both the single and segmented basin approaches and on Roseisle Creek.

The recorded and simulated flows for the three verification years are shown on Figures 3 to 5 for Roseisle Creek and on Figures 6 to 8 for the Boyne River upstream of Stephenfield. Figures 6 to 8 show simulated flows of the HSPF and SSARR models for the single basin approach only. The single basin and the segmented basin approach results for these models are compared separately on Figures 9 to 11.

A quantitative visual assessment of model performance is somewhat difficult. Therefore, a procedure was devised to enable a quantitative comparison of model performance. Model performance scores were computed based on two terms as follows:

$$Term\ 1 \quad \mid Q_s - Q_o \mid \div 2Q_o$$
$$Term\ 2 \quad \mid P_s - P_o \mid \div P_o$$

where: $Q_s$ = Simulated Mean Daily Flow
$Q_o$ = Observed Mean Daily Flow
$P_s$ = Simulated Peak Flow
$P_o$ = Observed Peak Flow

Term 1 is the sum of the daily absolute deviations between the simulated and observed flows divided by twice the sum of the observed flows. This term represents a combination of errors in simulated runoff volume and errors in timing between the simulated and recorded hydrographs. Term 2 is simply the absolute error in the forecasted peak expressed as a ratio of the observed peak. This method of computing performance scores was devised following the examination of various methods used in other studies (W.Q. Chin, 1980).

For both Term 1 and Term 2, the smaller the value the better the simulation. The sum of Terms 1 and 2 was subtracted from unity to produce the final performance score. A perfect simulation would produce a zero for both terms and the maximum attainable final score of 1.0.

Model performance scores for the verification years are summarized in Table 3. It is quite clear that the HSPF model performed the best of the four models for both Roseisle Creek and for the basin upstream of Stephenfield. For Roseisle Creek, the SSARR model placed second, followed by SLURP and MANAPI. For the basin upstream of Stephenfield, the MANAPI model had the second highest score, followed by SSARR and SLURP. Due to probable inaccuracies in input data and the fact that only three spring runoff events were used for verification, the performance figures should be treated with caution.

The performance scores shown in Table 3 indicate that there was no significant difference in the accuracy of simulations between the single basin and the segmented basin approaches.

For the HSPF model, performance scores for Roseisle Creek were considerably better than for the basin above Stephenfield. For the SSARR and SLURP models there was little difference in the performance on Roseisle Creek versus the basin above Stephenfield. The MANAPI model did not perform as well on Roseisle Creek as at Stephenfield.

Flows in 1969 were well simulated by the HSPF model and reasonably well simulated by the SSARR and SLURP models although the runoff volume was underestimated by the latter two models.

Flows in 1971 were well simulated by the HSPF and SSARR models and reasonably well simulated by the MANAPI model, using the single basin approach for all models. All models over-estimated runoff volume which suggested input data relating to initial basin soil moisture or basin snowpack may have been over-estimated.

Flows in 1974 for the single basin above Stephenfield were well simulated by the SLURP and MANAPI models and reasonably well simulated by HSPF. Flows on Roseisle Creek were well simulated by the HSPF model and reasonably well simulated by the three other models. Simulated flows on the rising limb were considerably higher than recorded flows for both the HSPF and SSARR models. Greater than average channel ice was reported in 1974 which could explain why simulated flows rose faster than actual flows.

There was little difference between models with respect to the accuracy of simulating the date of the peak flow. The MANAPI model appeared to be somewhat better than the other models in predicting flows resulting from rainstorms on the falling limb of the snowmelt runoff hydrograph, as occurred in 1971.

## Chapter 8

# EVALUATION OF HYDROLOGIC MODELS

The four hydrologic models were evaluated within six broad categories. The categories were: performance, cost of calibration, transferability, sensitivity to particular parameters, suitability for real-time forecasting and difficulty of model maintenance. Within these six categories, 25 factors were evaluated for the four models of interest. A summary of the model evaluation is shown on Table 4.

### 8.1 Performance of Models

As stated previously, the HSPF model was found to perform the best with respect to snowmelt runoff forecasting. The HSPF model also produced the best simulations of summer runoff as shown on Figure E-9. The latter comparison is based on the use of hourly precipitation data for the HSPF model and daily data for the other models. Summer calibrations of the HSPF model using daily data produced poor results.

### 8.2 Cost of Model Calibration

Time and computer costs required to calibrate a model for a typical watershed were calculated to be the highest for HSPF followed by SSARR, SLURP and MANAPI. It can be seen that the higher cost of the segmented basin approach over the single basin approach could not be justified on the basis of model performance. Further consideration was therefore not given to the segmented basin approach.

In the pilot project, spring runs with the HSPF and SSARR models used six-hourly data. Costs of running these models on daily data was, therefore, estimated to allow comparisons with the SLURP and MANAPI models. This would reduce model calibration costs by only 15 to 20 percent whereas the reliability of simulations would be significantly reduced under certain weather conditions as explained in Appendix E. The use of a six-hourly time interval for the HSPF and SSARR models thus appears to be cost effective.

## 8.3 Transferability

With respect to the transferability of the models, the chief concern is that data on solar radiation, evaporation and rainfall intensity will be less accurate in other watersheds which are further removed from point sources of these data. Networks for these meteorological elements are very sparse in southern Manitoba. Lack of sufficient high quality data is of greatest concern for the more complex models, HSPF and SSARR, which are particularly sensitive to these inputs.

A second concern involves the use of the models in other watersheds which have significantly different soils and topography. HSPF should have the advantage in this respect since it allows for the input of these physical parameters.

## 8.4 Model Sensitivity to Particular Input Data Parameters

The HSPF model is sensitive to nine input parameters, the SSARR model to eight parameters, the SLURP model to five parameters and the MANAPI model to four parameters. The most important parameters are listed in Table 4.

## 8.5 Suitability of Models for Real-Time Forecasting

The suitability of the various models for real-time forecasting was not fully evaluated since the HSPF, SSARR and SLURP models have not as yet been run in an operational situation. The more complex HSPF and SSARR models require more meteorological parameters than do the simpler SLURP and MANAPI models. Furthermore, the meaningful choice of meteorological and related inputs for the forecast period beyond present time would be more difficult for the HSPF and SSARR models. Model updating (tracking) could also be more difficult for the HSPF and SSARR models because there are more model parameters to adjust. For the experienced hydrologist however, this might not be a problem and the model flexibility could be used to advantage.

## 8.6 Model Maintenance

Time and cost involved in model maintenance is directly related to model complexity. Training time and model maintenance would be relatively high for HSPF, medium for SSARR and low for SLURP and MANAPI.

## 8.7 Discussion

The HSPF model performed the best of the four models tested. While it is one of the most physically-based models, it is still empirical in many respects. Calibration was difficult partially because input data was inexact. The SSARR, SLURP and MANAPI models are less expensive to calibrate and operate but produce poorer simulations, likely due to the parametric and index approaches they employ. An economic analysis showed that the superior performance of the HSPF model justified the higher cost of its usage. Economic justification is examined in the following chapter.

# Chapter 9

# ECONOMIC ANALYSIS OF HYDROLOGIC MODELS

An economic analysis was performed in order to determine whether the increased flow forecast accuracy of the HSPF model would justify the higher cost of calibrating and applying this model.

Since the SSARR and SLURP runoff models did not perform better than the presently used MANAPI model, there was no need to do a benefit/cost analysis of these two models. The economic analysis was therefore based on an estimate of the costs of Phase Two, and of the benefits associated with the replacement of the MANAPI procedures with HSPF forecasting on six key tributaries of the Red, Assiniboine and Souris River watersheds. The brief analysis indicated that Phase Two would have a benefit/cost ratio greater than one at the 5, 10 and 15 percent discount rates.

The incremental costs and benefits of Phase Two were estimated in 1983 dollars, based on a 20-year project life. Total costs over this period included cost outlays in Year 1 and Year 2, of $120,000 and $130,000, respectively and assumed annual operating and maintenance costs of $35,000 from Year 3 through Year 20. These costs included the salary of a hydrologist ($30,000) and computer costs ($5,000). These operating and maintenance costs are not covered in the cost-sharing agreement, but a complete benefit-cost analysis demands their inclusion.

The assessment of benefits of applying the HSPF model was evaluated from the perspective of tributary forecasting and flood routing of major floods on the Red, Assiniboine and Souris Rivers. Flood forecasts for the latest pre-melt outlook of past floods were first prepared using the present MANAPI model. Perfect forecasts were then superimposed for the six tributaries selected for HSPF application. The difference between the two forecasts would indicate the maximum improvement which would be attained through the implementation of Phase Two. A comparison of the two forecasts revealed that the maximum improvement in the forecasted spring peak stage would be about one-third of a metre (one foot) at Wawanesa on the Souris River and at Brandon and Holland on the Assiniboine River, and about one-sixth of a metre (one-half foot) at Morris and Ste. Agathe on the Red River.

Theoretical benefits of the improved flood forecasting system are generally described in several dimensions, such as, (1) increased accuracy in peak stage forecast, (2) increased accuracy in flow forecast, (3) increased lead time and (4) non-flood benefits in hydroelectric production, pollution control, recreation and some other water uses. The major improvement resulting from Phase Two would be in the peak stage forecast. In addition, Phase Two could increase the lead time for protective and preventative measures by about one day.

An attempt was made to establish a general relationship for the improved flood forecasting system benefits by expressing the percentage reduction in flood damages as a function of the increased accuracy in the peak stage forecast. The benefits were estimated for the Red River from Emerson to St. Norbert because information on damages and flood fighting costs for 1969, 1974, 1976 and 1979 were most readily available in this area. For the Assiniboine and Souris Rivers, data were insufficient to compute benefits, but it is known that damages are generally less than on the Red River. However, considering that the forecast improvement on these two rivers is estimated to be twice that for the Red River, the benefits on these two basins combined were assumed to be the same as the total benefits for the Red River. Thus, as a first approximation, calculated incremental benefits in terms of reduction in flood damages on the Red River were doubled to derive the total incremental benefits of Phase Two on the Red, Souris and Assiniboine Rivers.

The relevant frequency-damage and stage-damage data for the four years selected are documented in Table 5. From the frequency and damage data, the frequency-damage curve was drawn as shown on Figure 12. Since the maximum improvement in accuracy of the peak stage forecast was estimated to be plus or minus one-sixth metres, two new frequency-damage functions were then drawn, one-sixth of a metre above and below the appropriate stage coincident with a frequency of a given flood year. About one-half of the area covered under this band was used as a proxy of incremental benefits attributable to Phase Two. Only one-half of the area was used because the improved peak stage forecast would be either higher or lower, not both. Translated into average annual benefits, this amounted to $44,200 for the Red River and $88,400 for the Red, Souris and Assiniboine Rivers combined (double the Red River estimate). The incremental annual benefits for the Red River ($44,200) constituted only 5.1 percent of the total average annual damages ($869,200).

The preceding data on costs and benefits were used for the benefit/ cost analysis. Results are documented in Table 6. The benefit/cost ratios of Phase Two amounted to 1.83, 1.66 and 1.52 at the discount rates of 5, 10 and 15 percent, respectively. At the 10 percent discount rate, the net present value of costs was $499,000, the present value of benefits was $828,000, the payback period of this investment was four years and the internal rate of return was 58.3 percent.

The assumption about damage avoided was the weakest element in this analysis. However, it was tested to some extent by an examination of the worst case which has a benefit/cost ratio of 1.52. This showed that the assumed net present value of benefits could be reduced by 34 percent before the ratio would become one (i.e. from $636,000 to $420,000 - Table 6).

The responsiveness of the local population, governments and agencies to an improved flood forecasting and warning system is crucial. If a population is already responsive, it would normally be doing a great deal based on current forecasts thus reducing the scope for further "easy-to-come-by" benefits under the improved system. However, increased confidence in forecasts would lead to a greater assurance that what is preventable will be prevented. The reverse situation, with a history of poor local flood awareness and response, offers more scope for capturing incremental benefits, but a useful response would not be assured. The Manitoba situation resembles the first scenario.

# Chapter 10

# CHANNEL ROUTING STUDIES

Channel routing methodologies employed during Phase One studies included One-Dimensional Dynamic Routing (One-D), HSPF Kinematic Routing, SSARR Linear Reservoir Routing, Muskingum-Cunge Routing, and Muskingum Routing. These routing models are described in Appendix F.

Most of the work involved the testing of the One-D and Muskingum-Cunge routing methods on the Boyne River. The purpose of the channel routing studies was two-fold. First, main stem channel routing was required to compute flows for the segmented hydrologic modelling approach. This is necessary where the runoff at a downstream point of the segment is not been measured by a streamflow gauge. Secondly, channel routing studies were done to determine whether routing was necessary on tributaries and, if so, which routing method was best suited for tributaries.

It was demonstrated that the One-D method was unnecessarily complex and did not produce good results for small, relatively steeply-sloped tributary channels. Tests showed that there were significant differences in routed flows between the other four procedures. Further work is required to determine the most applicable procedure for tributary routing.

From the studies undertaken it is believed that the One-D dynamic routing method should have good success when applied to large rivers with shallow slopes such as the Red River. As a result, the Sub-Committee has engaged Mr. M. Sydor of the Water Planning and Management Branch, Environment Canada, to test the One-D model on the Red River from Emerson to Winnipeg during Phase Two. The Muskingum routing procedure is currently used on the Red, Assiniboine and Souris Rivers.

# Chapter 11

# REAL-TIME DATA NETWORKS

## 11.1 Introduction

The successful application of hydrologic simulation models for forecasting is dependent on the availability of real-time hydrometeorological data. Networks which can provide such real-time data are therefore a vital component of a successful flood forecasting system. In order to make accurate data such as river stages, precipitation and temperature available frequently and reliably, electronic means of data acquisition and transmission are required. Part of the Phase One studies therefore consisted of the installation, testing and evaluation of various types of field instruments and electronic data transmission systems.

## 11.2 Scope of Data Acquisition Studies

Four field data stations were installed and tested during Phase One. These were located on the Boyne River near Roseisle, on Stephenfield Reservoir, on Birdtail Creek near Birtle and on the Hespeler Floodway. All were equipped with water level instruments. Two stations were integrated, that is, meteorological instruments were colocated with hydrometric sensors. In order to evaluate different types of rain gauges, a Belfort weighing gauge was installed on the Boyne River near Roseisle and a tipping bucket rain gauge was installed on Birdtail Creek near Birtle.

Two methods of data transmission, satellite and land-line using the Data Collection Platform (DCP) and the Data Acquisition Telemetry System (DATS), were installed and tested. The DCP is an ultra high-frequency radio which transmits to a geostationary satellite which then retransmits to a receiving station on the ground. DATS has land-line based communications using the conventional telephone system. With a DCP, the user accesses the data from a storage and dissemination centre at the receive site, whereas with a DATS unit, the user accesses the data directly from the station by a telephone link. Both systems employ a standard computer terminal.

The operation of the instrumentation and data transmission system were evaluated during the year following installation. The value of the data acquisition systems for flood forecasting was to be evaluated based on operational use in spring flood events occurring during the study. Since no significant runoff events have taken place during the period since installation of the instruments, the evaluation of the data acquisition systems and their usefulness for flood forecasting was limited and preliminary.

## 11.3 Usefulness of Real-Time Data Systems for Flood Forecasting

Spring runoff in 1983 and 1984 on the Boyne River, Birdtail Creek and the Hespeler Floodway was relatively low and as a result there was little activity with regard to flood forecasting on these watersheds where the instrumentation had been placed. However, a few comments can be made based on the limited experience thus far.

i) The real-time electronic data stations have the ability to provide data much more frequently than can be obtained efficiently through gauge readers or telemarks. Thus there is much more information available with relatively minimal effort. The increased frequency of water level observations allow the interpretation of short term trends of flows at the gauging station. This capability allows updates of the forecasted flows earlier than was previously available using occasional water level readings. However, experience thus far has shown that during the active runoff period, water levels on tributaries fluctuate significantly over time intervals of several hours or less, probably due to a multitude of phenomenon. The flow forecaster therefore requires knowledge and experience in order to correctly interpret the meaning of the latest trend in the stage.

ii) Real-time electronic stations were found to be more reliable than conventional methods for obtaining real-time data. However, breakdowns in the equipment and/or interruptions in data transmission occurred from time to time. The possibility of a breakdown during a critical flood situation must be considered. It may be necessary to install manual staff or wire weight gauges at these sites and to have a gauge reader on standby in the event of a failure. The availability of a quick repair service would reduce the need for back-up or duplicate systems. An extended failure of a transmission system at an inopportune time would create serious difficulties.

iii) The availability of real-time data on precipitation and temperature in addition to water level data through the installation of these other sensors would be a definite asset to flood forecasting. The availability of rainfall intensity data from these stations would be particularly helpful. The Atmospheric Environment Service of

Canada presently operates only six rain gauges in the Manitoba portion of the Red, Assiniboine and Souris River watersheds which provide real-time data at time intervals of six hours or less. This network is inadequate to provide good areal coverage of short-term precipitation amounts on a real-time basis.

iv) The DCP system appeared to be superior to the DATS system based on a consideration of the performance of the instruments, on the data transmission systems, on the data requirements for flow forecasting and on cost. The DCP instrument has had a superior performance record and satellite data transmission has been more reliable than the land-line system. The three hour time interval between data transmissions with the DCP system was found to be generally adequate for flow forecasting. The somewhat higher initial capital cost of the DCP would be recovered in about three years by lower operating costs.

v) The Belfort weighing precipitation gauge was found to be more valuable for flow forecasting than the tipping bucket rain gauge because of its ability to accurately measure not only rain but also snow. Accurate estimation of the water content of snowfall is important for streamflow forecasting, particularly for late winter or spring snowstorms which typically have a high snow density.

Further detail on real-time data networks is contained in Appendix G.

# Chapter 12

# CONCLUSIONS AND RECOMMENDATIONS

Conclusions and recommendations are provided in response to the terms of the Agreement Respecting Flood Forecasting as set out below:

## 12.1 Phase One: Planning and Design

i) carry out a pilot project on the Boyne River for the purpose of developing a suitable deterministic runoff model for use on the Red, Assiniboine and Souris Rivers in Manitoba,

ii) evaluate the need for, and where necessary, arrange for installation of appropriate equipment to obtain additional field data required for carrying out the Boyne River pilot project,

iii) carry out such tasks as may be required to design, develop, calibrate, test and prepare, and document formal procedures for an adequate flood forecasting system for the Red, Assiniboine and Souris Rivers of Manitoba,

iv) establish and staff a Canada-Manitoba flood forecasting centre to be located in the offices of the Manitoba Department of Natural Resources for the Red, Assiniboine and Souris Rivers in Manitoba, and

v) undertake tasks related or necessarily incidental to the above.

## 12.2 Phase Two: Development and Operation

Upon conclusion of Phase One, the Parties shall decide whether to proceed with Phase Two based on a recommendation and report from the Steering Committee.

The results of the model comparisons indicated that although present forecasting procedures provide reasonable results, benefits could be gained by applying a more physically-based model. The HSPF model performed better than the existing MANAPI model in the verification years. Also a statistical model like that currently used can only be used with confidence when input data are in the range of the historical values used for development. Physically-based models should provide more dependable results when meteorological and upstream flow conditions are outside the range of those used in model calibration. Of the deterministic and parametric models tested in the Pilot Project the SLURP model was least accurate. The model was considered to be too simplistic. The HSPF model performed better than the SSARR model and is a more "state of the art" model. It therefore holds more promise for improved performance with increased experience of the modeller. The HSPF model costs more to calibrate and to run than the SSARR model. However, the Water Resources Branch recently purchased a mini-computer. With an in-house mini-computer the cost comparisons become of limited importance. The HSPF model would take a little more time to calibrate for each basin than the SSARR model.

**It is recommended that the HSPF model be selected for application in Phase Two.**

It was shown that the One Dimensional Dynamic (One-D) routing model is not suitable for channel routing on smaller, more steeply sloped tributaries such as the Boyne River. Of the remaining four channel routing models investigated for tributary routing, no significant technical advantages have yet been identified for using one model over another. However, these studies are not yet complete. There is evidence that the One-D routing model has some advantages over the other routing models on the main stem rivers.

**It is recommended that the routing model that is easiest to use with the recommended hydrologic model should be applied to tributary routing in Phase Two unless further work with the routing models indicates that one of the other models is superior. Also an attempt should be made to apply the One-D model to the main stem rivers.**

The MANAPI forecasting procedures which Manitoba has developed during the 1970's have been shown to perform reasonably well, although some refinements are required. MANAPI will continue to be used on tributaries where HSPF has not been applied. Documentation of MANAPI forecasting procedures has not been completed.

**It is recommended that refinements to Manitoba's current MANAPI forecasting procedures be made during Phase Two and documentation of these procedures be made.**

The four real-time hydrometeorologic data stations installed during Phase One have demonstrated the reliability of electronic field instruments and of real-time data acquisition by satellite. Furthermore, the application of the HSPF model will require radiation and rainfall intensity data in some areas where the data is not currently available.

**It is recommended that installation of field equipment that can record**

**and transmit radiation and rainfall as well as water levels, precipitation and temperature be done where feasible and as the needs dictate.**

A Canada-Manitoba Flood Forecasting Centre was established in the offices of the Manitoba Water Resources Branch. Two professional engineers and one technologist were hired to perform studies under this Agreement.

**It is recommended that the Canada-Manitoba Flood Forecasting Centre be made operational during Phase Two.**

An economic analysis carried out in Phase One demonstrated that the benefit/cost ratio of integrating a physically-based model into flood forecasting in southern Manitoba would have a benefit/cost ratio of 1.83, 1.66 and 1.52 at discount rates of 5, 10 and 15% respectively. While financial and time constraints inherent in Phase Two prohibit a general application of the HSPF model to all of the approximately 40 sub-basins in the Red, Assiniboine and Souris Rivers in Manitoba, the economic analysis shows that application of the model to six selected tributaries would result in a benefit/cost ratio greater than one.

**It is recommended that Phase Two be undertaken and that the HSPF model be initially applied to selected tributary basins which are known to be particularly important for the forecasting of peak flows on the Red, Assiniboine and Souris Rivers, or in which the present statistical method has produced poorer than average results. For the other tributary basins the MANAPI method will continue to be applied until such time as the HSPF model can be applied and shown to provide an improvement in the flood forecasting system.**

# Bibliography

[1] Blaney, H.F. and Criddle, W.D., 1950. *Determining Water Requirements in Irrigated Areas from Climatological and Irrigation Data.* U.S.D.A.-SCSTP96. Washington, D.C.

[2] Chin, W.Q., 1981. *Operational Flood Forecast Models - Selection and Comparison.* Hydrologic Applications Division, Department of the Environment, Canada.

[3] *Conservation and Flood Control, Assiniboine River.* 1952. Prairie Farm Rehabilitation Branch, Department of Agriculture, Canada.

[4] Kite, G.W., 1976. *Development of a Hydrologic Model for a Canadian Watershed.* Water Resources Branch, Environment Canada.

[5] Kohler, M.A., Nordenson, T.J. and Fox, W.E., 1955. *Evaporation from Pans and Lakes.* U.S. Weather Bureau Research Paper No. 38.

[6] Meyer, A.F., 1942. *Evaporation from Lakes and Reservoirs.* Minnesota Resources Commission, St. Paul, Minnesota.

[7] *Report on Investigations into Measures for the Reduction of the Flood Hazard in the Greater Winnipeg Area, 1953. Appendix "C", Flood Runoff Analysis.* Red River Basin Investigation, Water Resources Division.

[8] *Snow Hydrology, 1956.* North Pacific Division Corps of Engineers, U.A. Army, Portland, Oregon.

[9] Warkentin, A.A., 1973. *Runoff Forecasting in the Boyne River Watershed Upstream of Carman, Manitoba (unpublished).*

## Table 1
## Hydrologic Models Given Cursory Consideration

| Model Name | Developed by |
|---|---|
| HEC-1 | Hydrologic Engineering Centre |
| HYMO | U.S. Soil Conservation Service |
| *HSPF | U.S. EPA, Hydrocomp |
| ILLUDAS | Illinois State Water Survey |
| MITCAT | Resource Analysis, Massachusetts |
| MMDW | Agricultural Eng., U. of Minnesota |
| RROUT | CH2M HILL Inc. |
| *SSARR-4 | U.S. Army Corps. of Engineers |
| STORM | U.S. Army Corps. of Engineers |
| USDAHL | USDA Hydrographic Laboratory |
| *NWSRFS | U.S. National Weather Service |
| *SASK-6 | UBC, Canada Government Agencies |
| *SLURP | Canada Water Resources |
| *MANTHORN | Manitoba Water Resources Branch |
| *QUFM | Queens University, Ontario |
| *MANAPI | Manitoba Water Resources Branch |
| UBC | University of British Columbia |
| PARAMETRIC | Shawinigan Engineering |

*Models selected for more detailed analysis

## Table 2
## Model Selection Criteria

### Primary Model Characteristics (Minimum Requirements)

A. Basic Model Structure

1) Uses physically based conceptual algorithms.
2) Is continuous-in-time (not event) at least for the spring runoff period.
3) Uses FORTRAN programming language, suitable to run on IBM or CDC computers.
4) Operates on a time step of from one hour to 24 hours depending on the model component.
5) Uses a multiple catchment approach rather than a lumped basin.
6) Is suitable for use on large river basins.
7) Is suitable for use in operational flood forecasting.

B. Data Management

1) Data requirements commensurate with that generally available within the Red, Assiniboine and Souris River basins.
2) Flexible and efficient data management system for handling time series of historical data.

C. Soil Moisture

1) Soil moisture modelled on a daily basis during the spring runoff period.
2) Soil moisture at freeze-up determined from a daily or monthly soil moisture budget computed from observed precipitation and evaporation data using parametric methods.

D. Infiltration

1) Infiltration rate related to soil moisture and soil type.
2) Effect of frozen soil at least estimated.

E. Snowmelt

1) Simplified energy budget used, with modelling of sensible heat transfer, radiation melt, condensation melt.
2) Melt calculations performed on a 6-hour interval basis.

F. Water Budget

1) Surface depression storage accounted for at least implicitly.

    2) Evaporation losses may be ignored during the spring runoff period.
    3) Interflow modelled but deep percolation ignored.

G. Conveyance

    1) Does not use unit hydrographs.
    2) Overland flow handled as unsteady flow with slope, length of flow path, and surface roughness considered.
    3) Dynamic channel routing used, with explicit evaluation of channel storage.

H. Complexity

    1) Costs of mobilization and calibration within limits of proposed time and cost of study.
    2) Cost of operational use of model within reason.

## Secondary Model Characteristics (Optimum Condition)

1. Time step for calculations is flexible in all model components.

2. Groundwater contribution to streamflow included.

3. Soil moisture modelled throughout the summer on a daily basis.

4. Actual evapotranspiration computed from potential evaporation using empirical formulae.

5. Where pan evaporation is not available, potential evapotranspiration computed from air temperature and vapour pressure, wind, and sunshine.

6. Snowmelt computations include detailed energy budget, with estimates of net radiation, snow albedo, percent snow cover, etc.

7. Surface depression storage handled explicitly.

8. Infiltration rates related empirically to soil type, soil moisture, soil temperature and land use.

9. Model has demonstrated capability of reproducing historic flood events.

10. Model is being used by other flow forecasting agencies.

## Table 3
## Hydrologic Model Performance Scores For Verification Years

| Model | Basin Upstream of Stephenfield |  | Roseisle Creek |
|---|---|---|---|
|  | Single Basin Approach | Segmented Basin Approach |  |
| HSPF |  |  |  |
| 1969 | 0.76 | 0.86 | 0.88 |
| 1971 | 0.83 | 0.74 | 0.96 |
| 1974 | 0.65 | 0.56 | 0.88 |
| Mean | 0.75 | 0.72 | 0.91 |
| SSARR |  |  |  |
| 1969 | 0.62 | 0.58 | 0.62 |
| 1971 | 0.80 | 0.75 | 0.70 |
| 1974 | 0.38 | 0.40 | 0.63 |
| Mean | 0.60 | 0.58 | 0.65 |
| SLURP |  |  |  |
| 1969 | 0.62 | Not Applicable | 0.56 |
| 1971 | 0.26 | " | 0.61 |
| 1974 | 0.71 | " | 0.61 |
| Mean | 0.53 | " | 0.59 |
| MANAPI |  |  |  |
| 1969 | 0.56 | Not Applicable | 0.45 |
| 1971 | 0.66 | " | 0.57 |
| 1974 | 0.72 | " | 0.62 |
| Mean | 0.65 | " | 0.55 |

# TABLE 4

| CRITERION | HSPF | | SSARR | | SLURP | MANAPI |
|---|---|---|---|---|---|---|
| 1. PERFORMANCE SCORES BASED ON SNOWMELT RUNOFF FORECASTS FOR VERIFICATION YEARS | Single Basin | Segmented Basin | Single Basin | Segmented Basin | | |
| a) Boyne River at Stephenfield | | | | | | |
| i) $\Sigma \lvert Q_s - Q_o \rvert \div 2 \Sigma Q_o$ | 0.17 | --- | 0.20 | 0.24 | 0.24 | 0.17 |
| ii) $\lvert P_s - P_o \rvert \div P_o$ | 0.09 | --- | 0.20 | 0.18 | 0.23 | 0.18 |
| 1 - (i + ii) | 0.75 | 0.72 | 0.60 | 0.58 | 0.53 | 0.65 |
| b) Roseisle Creek | | | | | | |
| i) $\Sigma \lvert Q_s - Q_o \rvert \div 2 \Sigma Q_o$ | 0.14 | --- | 0.22 | --- | 0.23 | 0.26 |
| ii) $\lvert P_s - P_o \rvert \div P_o$ | 0.08 | --- | 0.13 | --- | 0.18 | 0.20 |
| 1 - (i + ii) | 0.91 | --- | 0.65 | --- | 0.59 | 0.55 |
| c) Timing of Spring Peak at Stephenfield (Error in Days) | zero to one | --- | zero to two | zero to two | zero to one | zero to one |
| d) Accuracy of Spring Rainfall Runoff Forecasts | fair | --- | poor | poor | fair | fair to good |
| e) Accuracy of Summer Runoff Forecasts | fair to good | --- | fair | --- | poor | fair |
| 2. COST OF CALIBRATION FOR A TYPICAL 1000 KM² WATERSHED ($ 1983 Canada) | | | | | | |
| - climatological data preparation using daily data where possible | 2500 | 3500 | 2000 | 3000 | 1000 | 50 |
| - climatological data preparation using hourly and synoptic data | 3000 | 4000 | 2500 | 3500 | --- | --- |
| - model calibration (daily data) | 1300 | 4000 | 500 | 1200 | 500 | 50 |
| - model calibration (synoptic data) | 1500 | 4500 | 700 | 1600 | --- | --- |
| - channel routing | 500 | 500 | 200 | 200 | NIL | NIL |
| - total cost using daily data | 4300 | 8000 | 2500 | 4400 | 1500 | 100 |
| - total cost using hourly and synoptic data | 5000 | 9000 | 3200 | 5300 | --- | --- |
| - time using daily data (man-mo.) | 2.5 | 5.5 | 1.6 | 3.5 | 1.0 | 0.5 |
| - Time using hourly and synoptic data (man-mo.) | 3.0 | 6.5 | 2.0 | 4.3 | --- | --- |
| 3. MODEL TRANSFERABILITY | | | | | | |
| a) Climatological Data Constraints | Radiation, Evaporation and Rainfall Intensity would be poorer in most other basins | | Radiation and Evaporation data would be poorer in most other basins | | Evaporation data would be poorer in most basins | Data elsewhere about the same as for the Boyne above Stephenfield |
| b) Watershed Feature Constraints | Would require new conceptualization in some basins | | Would require new runoff and loss curves for most basins | | Parameters would be adjusted as best possible | Parameters are optimized to suit watershed conditions. |
| c) Channel Routing Constraints | A survey might be required to obtain channel cross-sections | | Travel times might have to be measured in some cases | | Lack of channel routing is a drawback of the model | Unit hydrographs are used. |
| 4. INPUT PARAMETERS WHICH THE MODEL IS PARTICULARLY SENSITIVE TO | Inputs for snowmelt equation, Rainfall Amount and Intensity, Snowpack, initial storages | | Inputs for snowmelt equation, Soil Moisture Index, Snowpack, Rainfall Amount | | Snowpack, Melt Factor | Snowpack, Antecedent Precipitation Index. Melt Index for peak forecast |
| 5. MODEL SUITABILITY FOR REAL-TIME FLOW FORECASTING | | | | | | |
| a) Data Availability | Radiation data would be difficult to obtain | | Radiation data would be difficult to obtain | | All data elements are available | All data elements are available |
| b) Ease of Data Input | Easy for known data but difficult for probabilistic data | | Easy for known data but difficult for probabilistic data | | Relatively easy but some work required | Relatively easy |
| c) Ease of Model Updating (tracking) | Not difficult as to the mechanics | | Not difficult as to the mechanics | | Not difficult as to the mechanics | Not difficult as to the mechanics |
| d) Computer Cost per Tributary (for a single spring run) | Approximately \$12 | | Approximately \$5 | | Approximately \$3 | Approximately \$0.50 |
| e) Training Time | High | | Medium | | Low | Very Low |
| 6. MODEL MAINTENANCE | Not difficult for someone familiar with the model | | Not difficult | | Very easy | Very easy |

## TABLE 5

### BASIC DATA FOR DEVLEOPING DAMAGE-FREQUENCY CURVES
### RED RIVER VALLEY

| Years of Major Floods | Frequency (%) | Stage (Feet) | Damages (actual $)* (million) | Damages (1983 $)* (million) |
|---|---|---|---|---|
| 1976 | 22.0 | 767.94 | 0.00 | 0.00 |
| 1969 | 13.0 | 774.04 | 0.20 | 0.60 |
| 1974 | 9.8 | 776.24 | 1.12 | 2.23 |
| 1979 | 3.3 | 781.26 | 11.30 | 14.85 |
| ---- | 1.0 | 785.15 | --- | 20.00 |

*Damages in current dollars were converted to 1983 dollars by using Statistics Canada's Consumer Price Index: 1971 = 100.

## TABLE 6

### PHASE TWO IMPROVED FLOOD FORECASTING SYSTEM
### QUANTITATIVE BENEFIT-COST RESULTS

| Benefit-Cost Parameters | Discount Rates<br>5% | 10% | 15% |
|---|---|---|---|
| Benefit-Cost Ratios | 1.83 | 1.66 | 1.52 |
| Net Present Value of Costs | $ 633,000 | $499,000 | $420,000 |
| Net Present Value of Benefits | $1,157,000 | $828,000 | $636,000 |
| Present Value of Net Benefits | $ 523,000 | $329,000 | $217,000 |
| Pay-Back Period | 4 years | 4 years | 4 years |
| Internal Rate of Return | | 58.3% | |

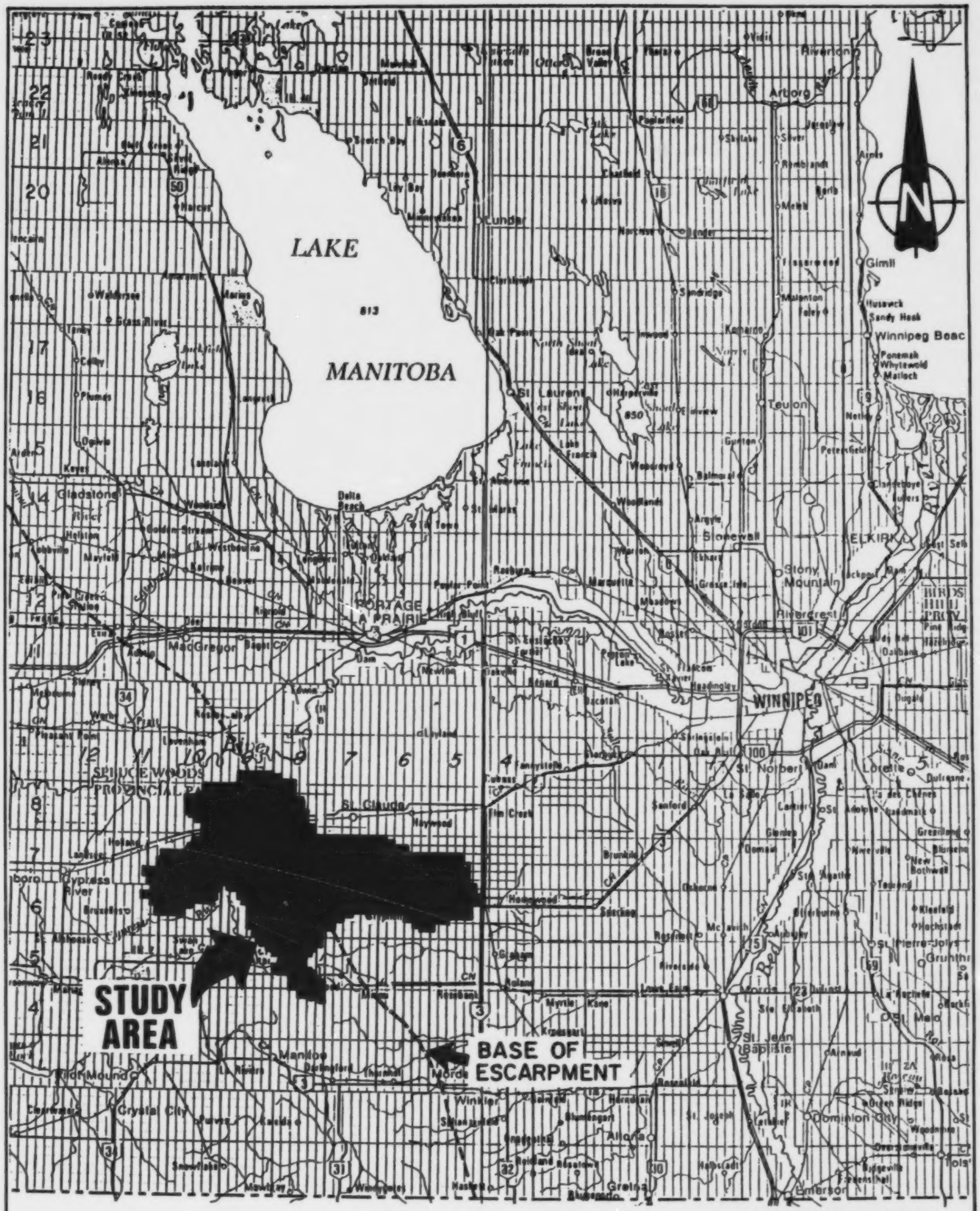

LAKE
MANITOBA
N
STUDY AREA
BASE OF ESCARPMENT
PORTAGE LA PRAIRIE
WINNIPEG
SELKIRK
Gimli
Winnipeg Beac
Stonewall
Stony Mountain
Teulon
St Laurent
Lundar
Gladstone
MacGregor
Spruce Woods Provincial Park
St Claude
Cypress River
Carman
Morris
Morden
Winkler
Manitou
Pilot Mound
Crystal City
Altona
Emerson
Dominion City
St Jean Baptiste
St Norbert
Riverton
Arborg
12
11
10
9
8
7
6
5
4

**FIGURE 1**

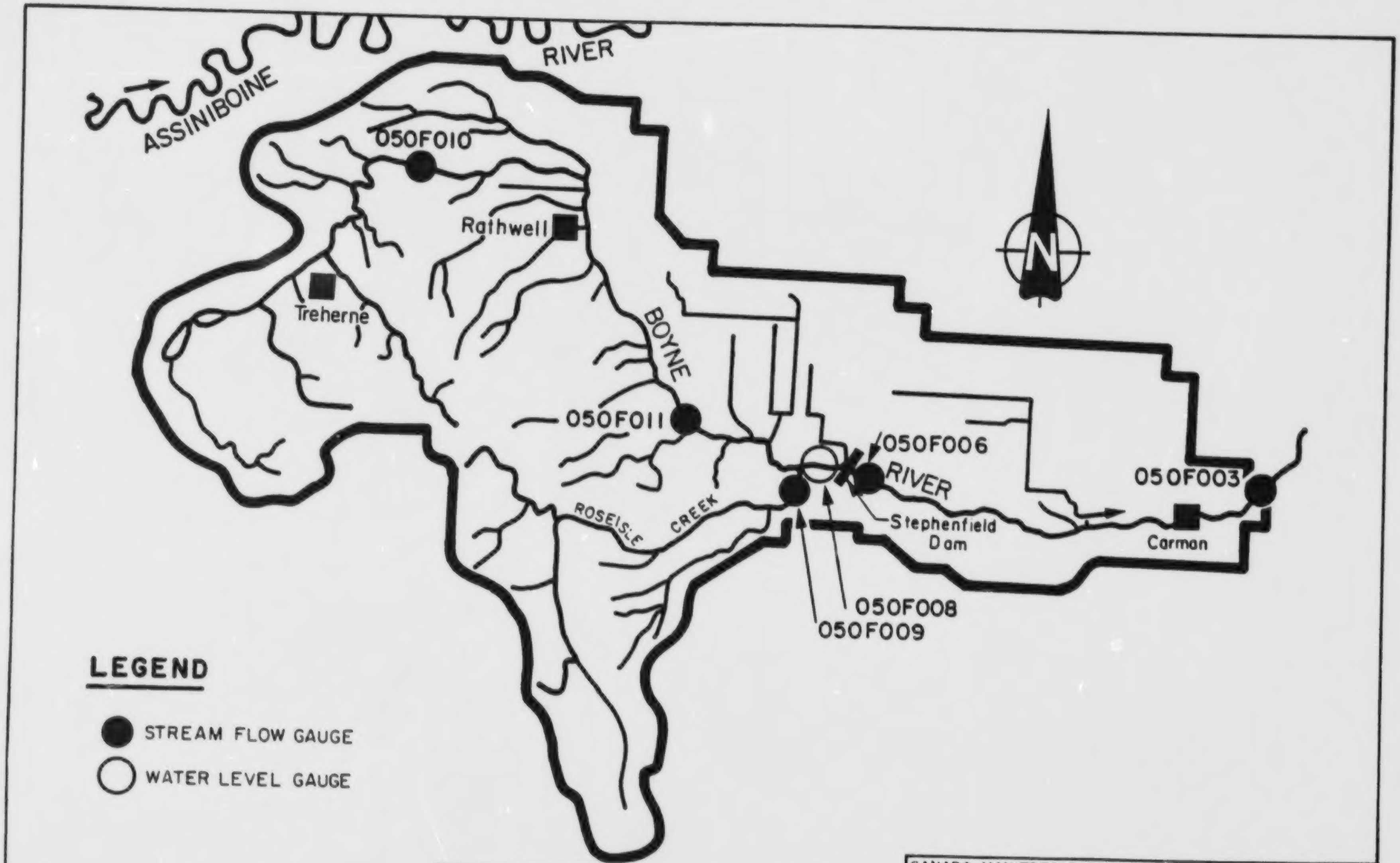
CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

## BOYNE RIVER WATERSHED
## LOCATION PLAN

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>A.A.W. | DRAWN J.M.<br>CHECKED | SUBMITTED | APPROVED | SCALE<br>AS SHOWN | DATE<br>84 05 22 | SHEET<br>OF | FILE NO. |


ASSINIBOINE
RIVER
050F010
Rathwell
Treherne
BOYNE
050F011
ROSEISLE
CREEK
050F006
RIVER
Stephenfield Dam
050F008
050F009
050F003
Carman
N
LEGEND
● STREAM FLOW GAUGE
○ WATER LEVEL GAUGE

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WATER RESOURCES BRANCH

CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

BOYNE RIVER WATERSHED
DRAINAGE & HYDROMETRIC NETWORK

| | | | |
|---|---|---|---|
| PREPARED A. A.W. | DRAWN J. M. | SUBMITTED | APPROVED |
| | CHECKED | | |

| | | | |
|---|---|---|---|
| SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

FIGURE 2

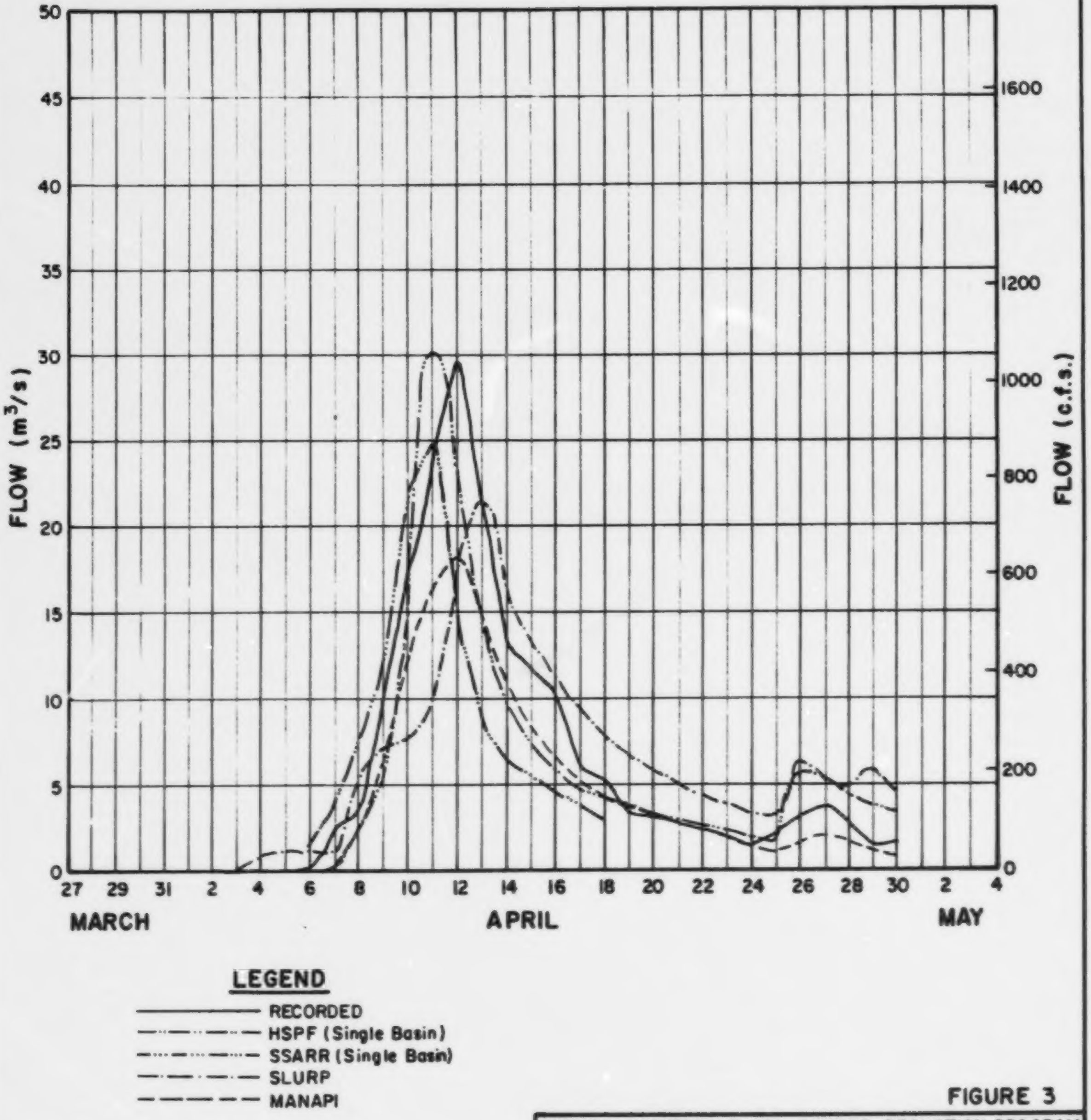

FIGURE 3

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

SIMULATED AND RECORDED FLOWS
ROSEISLE CREEK NEAR ROSEISLE
VERIFICATION - SPRING OF 1969

| PREPARED | DRAWN C.E.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |
|---|---|---|---|---|---|---|---|
| E.L.B. | CHECKED A.W. | | | | | | |

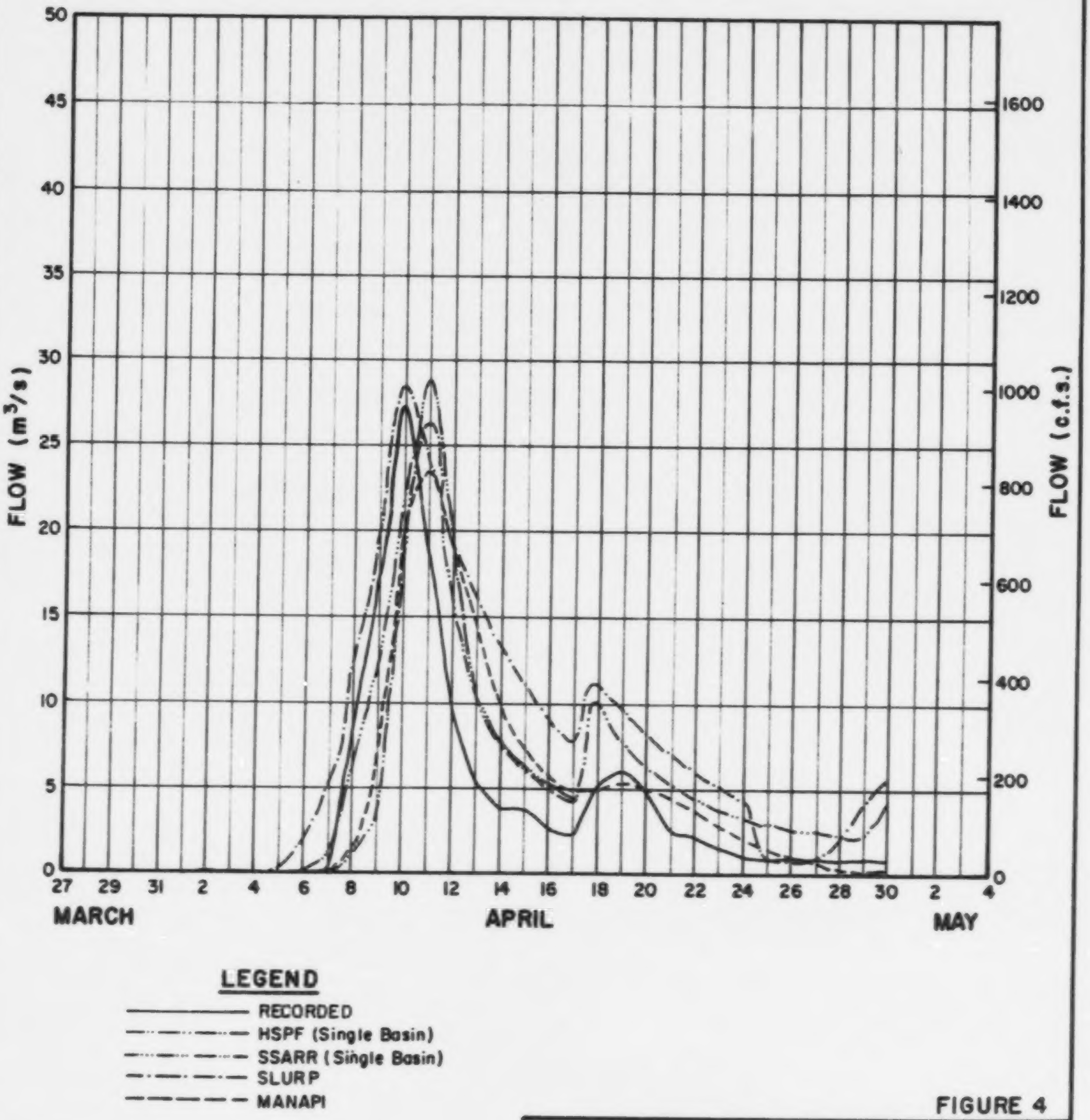

FIGURE 4

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

SIMULATED AND RECORDED FLOWS
ROSEISLE CREEK NEAR ROSEISLE
VERIFICATION - SPRING OF 1971

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>E. L. B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

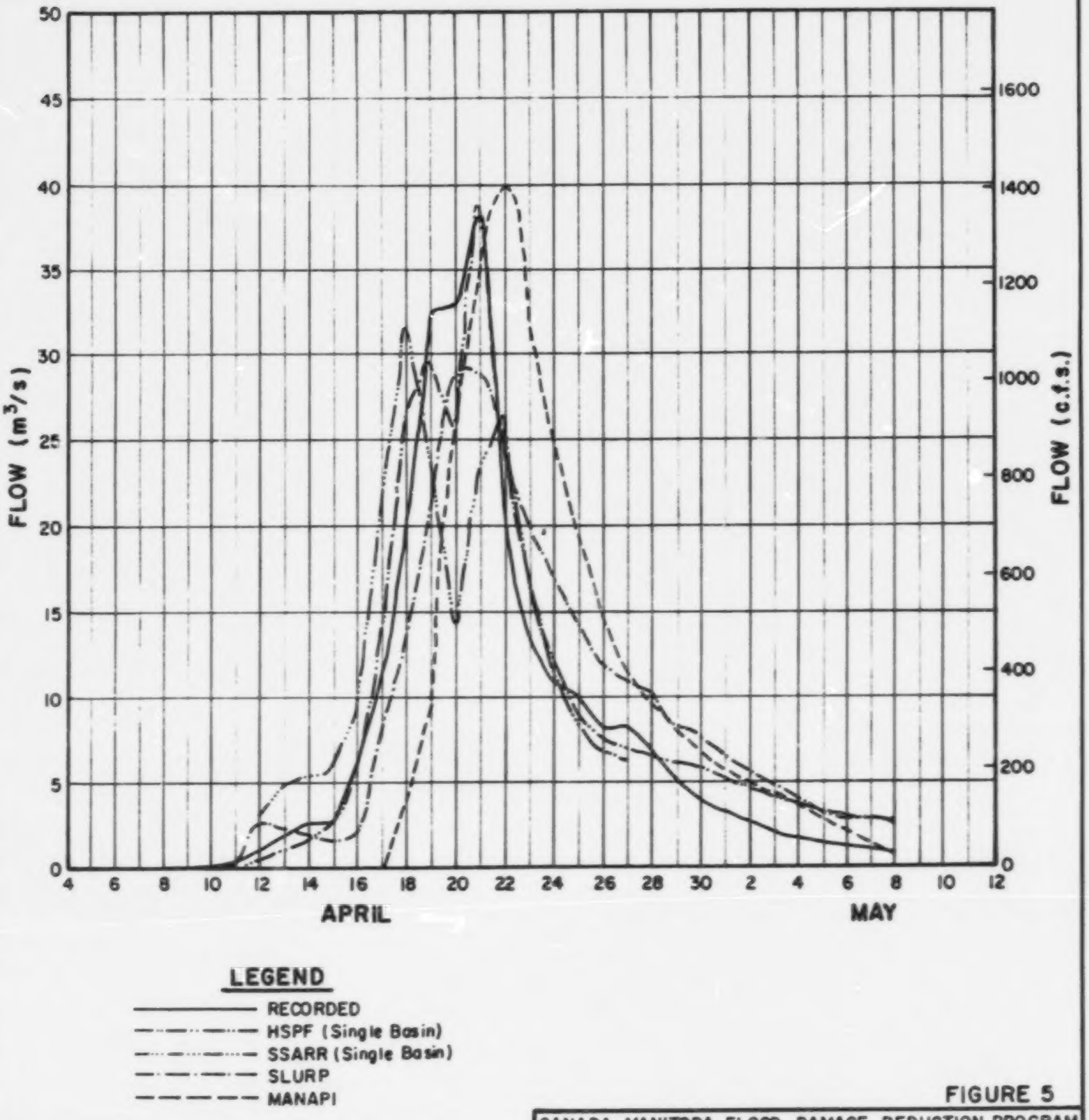

FIGURE 5

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT -PHASE ONE

## SIMULATED AND RECORDED FLOWS ROSEISLE CREEK NEAR ROSEISLE VERIFICATION - SPRING OF 1974

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>E.L.B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

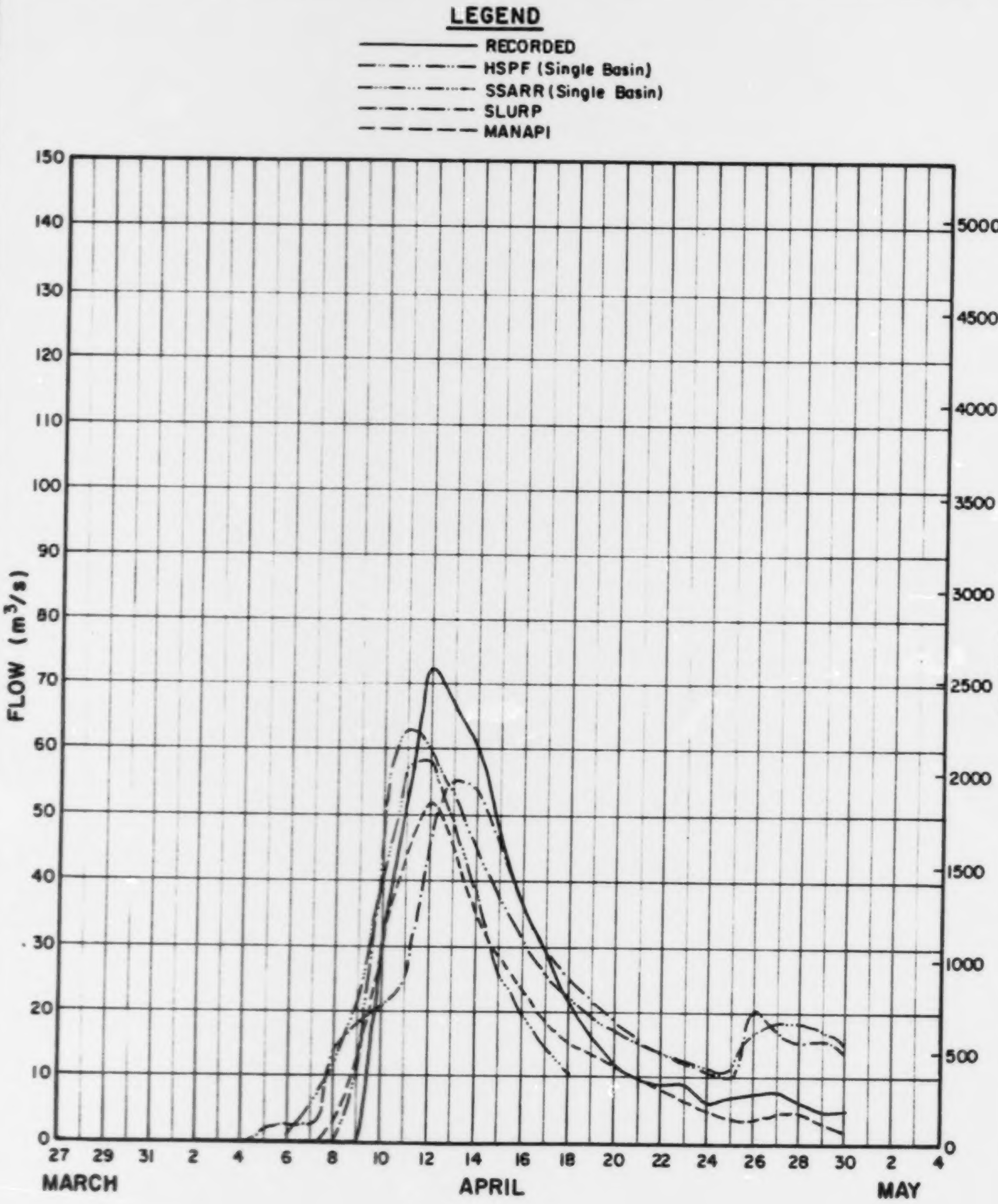

FIGURE 6

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## SIMULATED AND RECORDED FLOWS
## BOYNE RIVER NEAR STEPHENFIELD
## VERIFICATION - SPRING OF 1969

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED E.L.B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

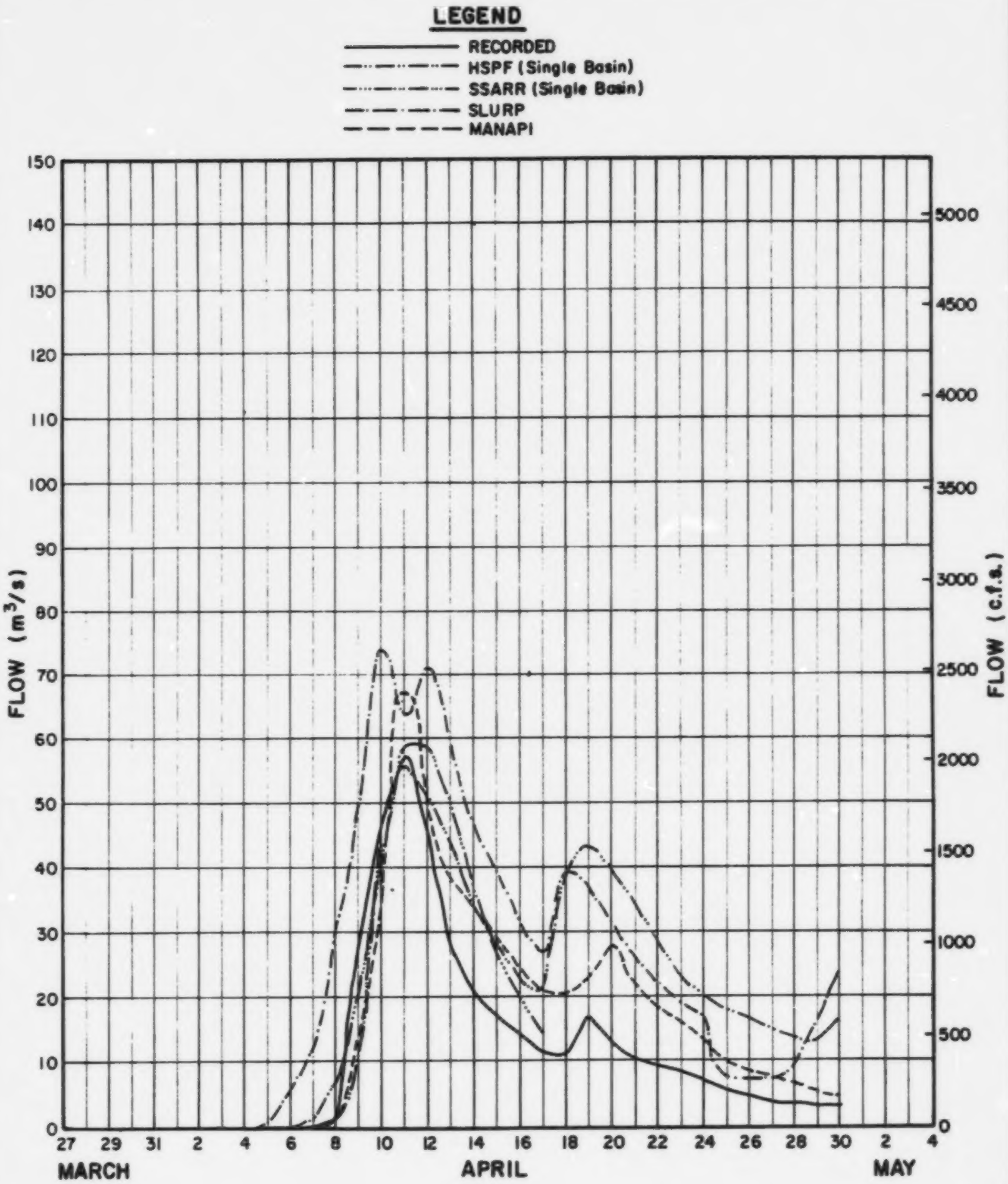

FIGURE 7

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

## SIMULATED AND RECORDED FLOWS BOYNE RIVER NEAR STEPHENFIELD VERIFICATION - SPRING OF 1971

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| E.L.B | C.E.W.<br>CHECKED A.W. | | | AS SHOWN | 84 05 22 | OF | |

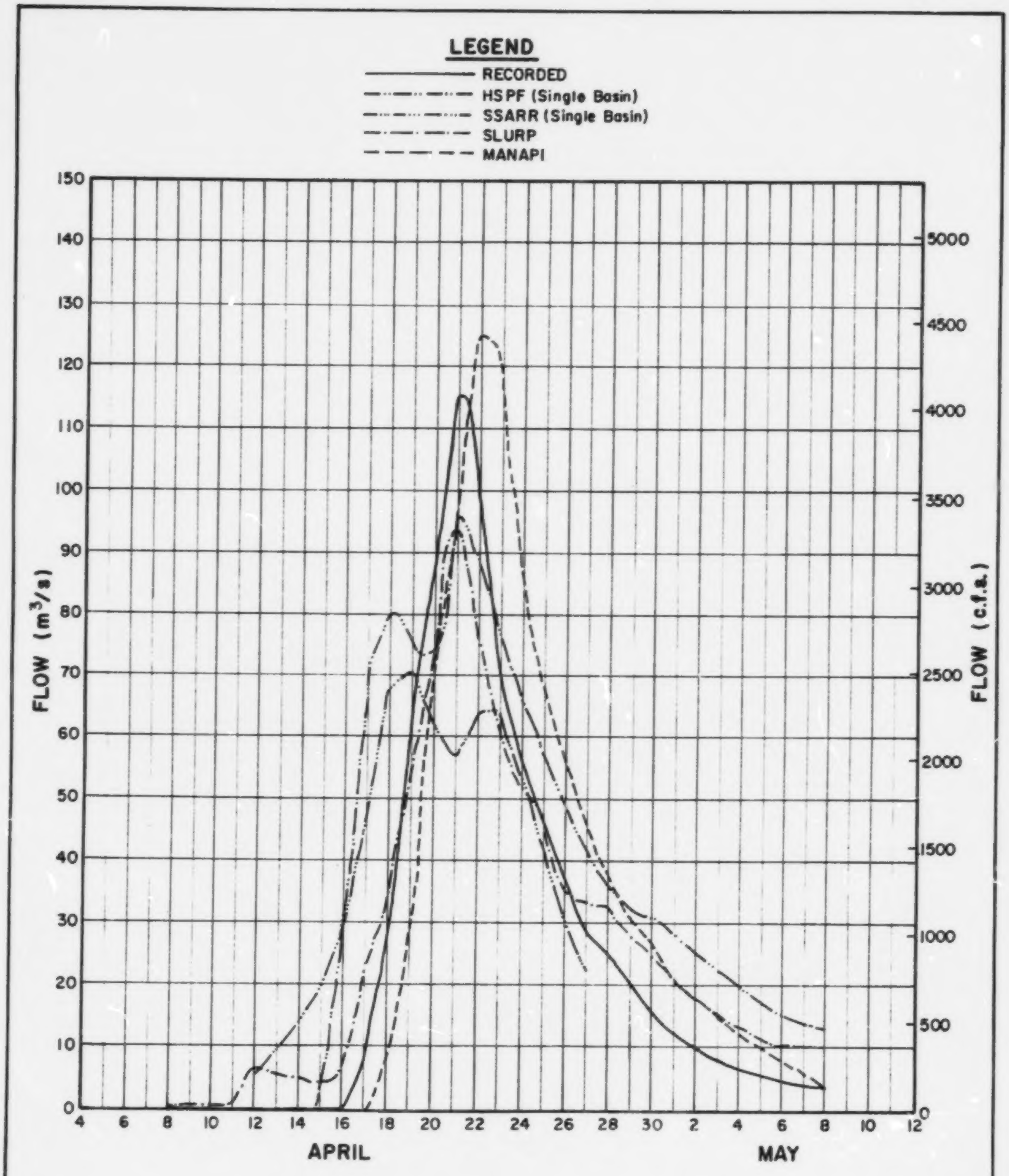

FIGURE 8

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## SIMULATED AND RECORDED FLOWS
## BOYNE RIVER NEAR STEPHENFIELD
## VERIFICATION - SPRING OF 1974

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>E.L.B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

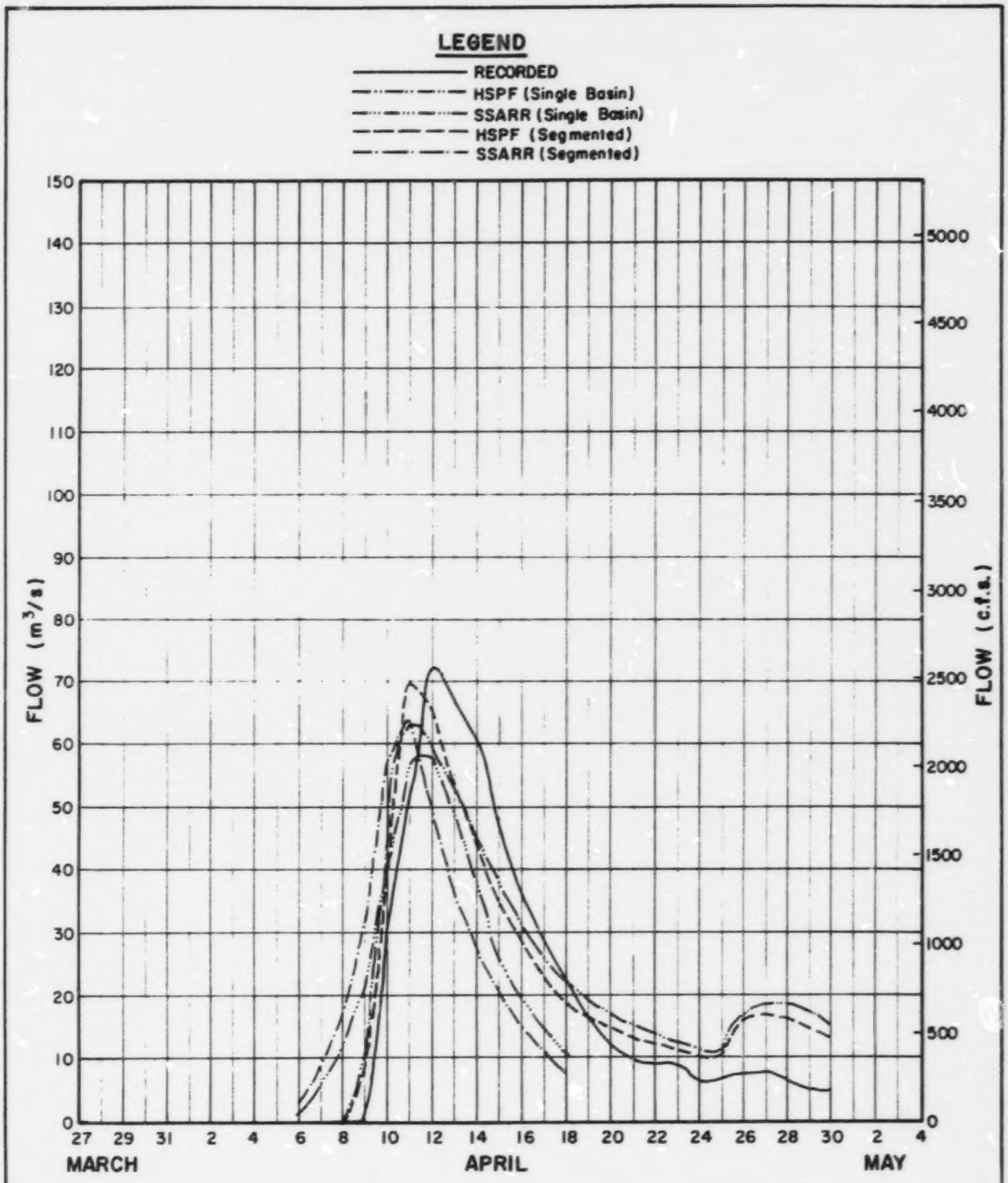

FIGURE 9

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

# SIMULATED AND RECORDED FLOWS BOYNE RIVER NEAR STEPHENFIELD SINGLE VS. SEGMENTED BASIN APPROACH VERIFICATION - SPRING OF 1969

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>A.W. | DRAWN C.E.W.<br>CHECKED | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

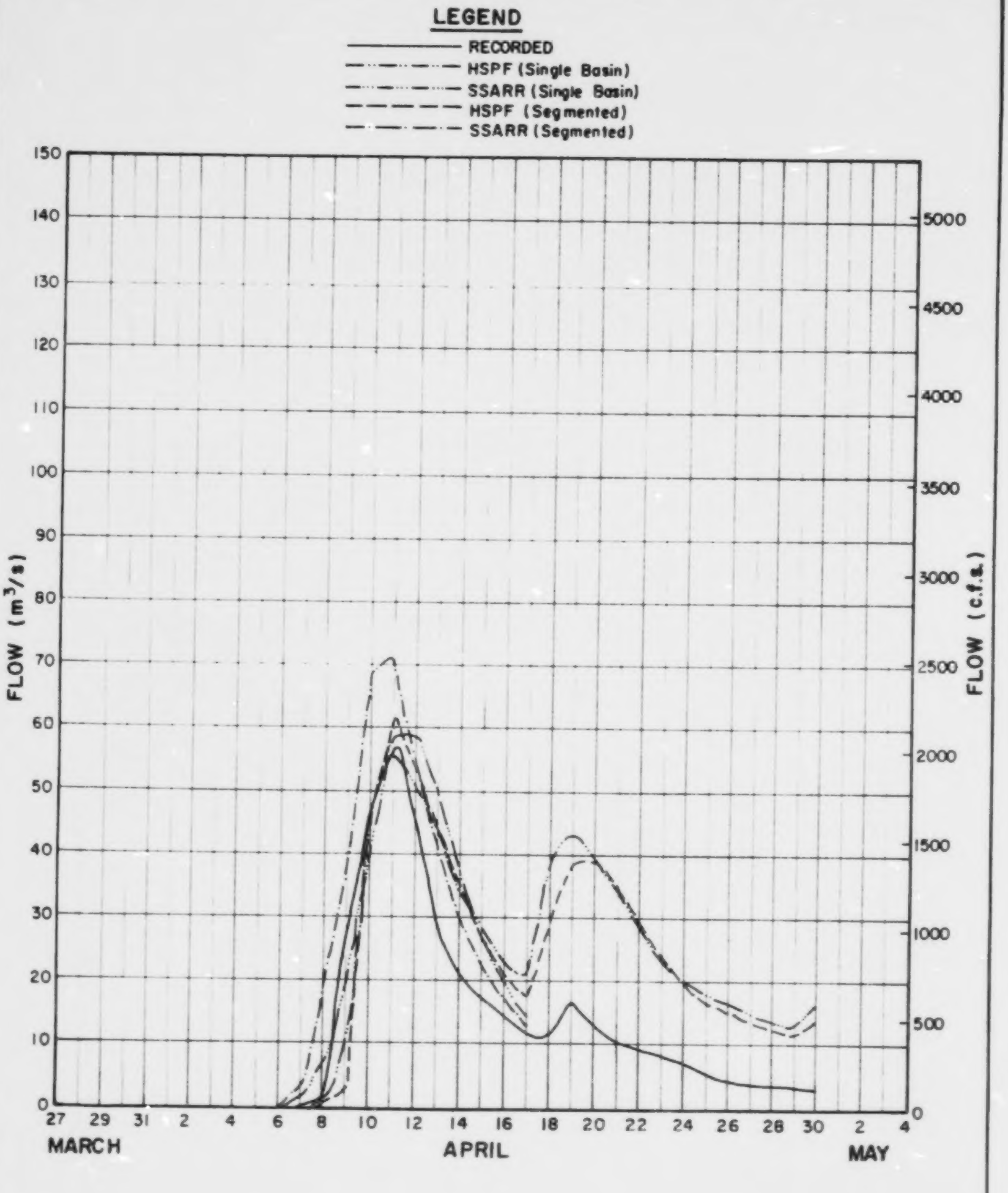

FIGURE 10

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## SIMULATED AND RECORDED FLOWS BOYNE RIVER NEAR STEPHENFIELD SINGLE VS. SEGMENTED BASIN APPROACH VERIFICATION - SPRING OF 1971

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>A.W. | DRAWN C.E.W.<br>CHECKED | SUBMITTED | APPROVED | SCALE<br>AS SHOWN | DATE<br>84 05 22 | SHEET<br>OF | FILE NO. |

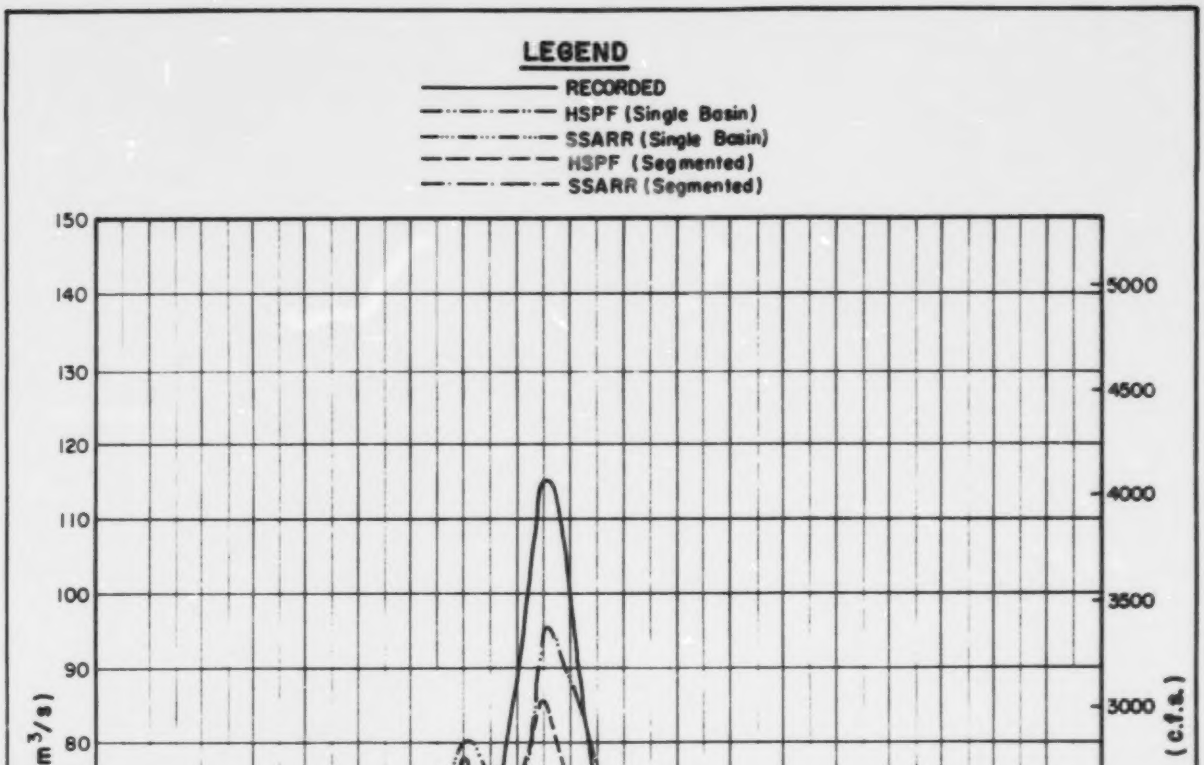

FIGURE 11

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

SIMULATED AND RECORDED FLOWS
BOYNE RIVER NEAR STEPHENFIELD
SINGLE VS. SEGMENTED BASIN APPROACH
VERIFICATION - SPRING OF 1974

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| A.W. | C.E.W.<br>CHECKED | | | AS SHOWN | 84 05 22 | OF | |

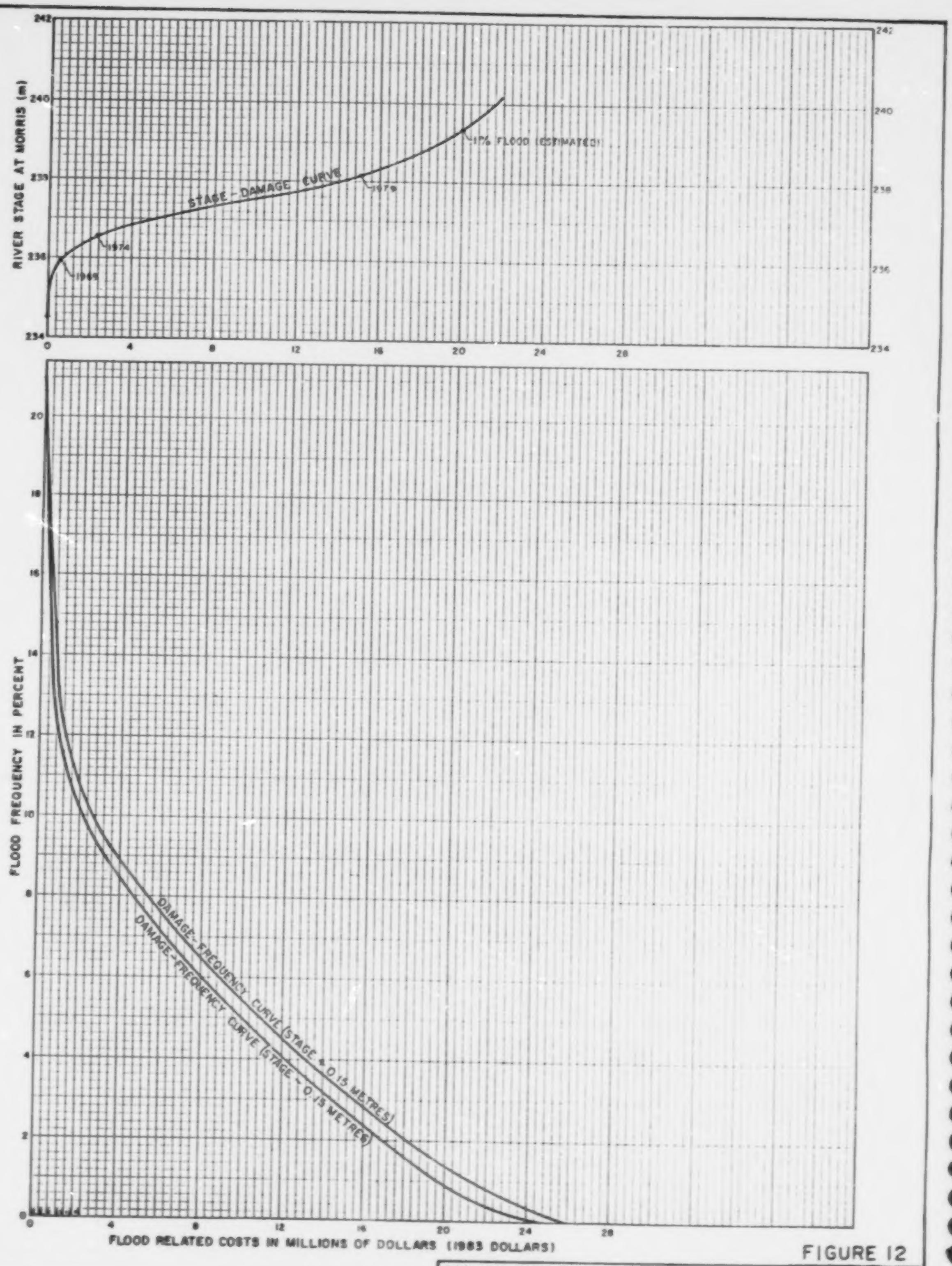

FIGURE 12

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## STAGE-DAMAGE & FREQUENCY-DAMAGE CURVES FOR RED RIVER FROM EMERSON TO ST. NORBERT

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED A.A.W. | DRAWN C.E.W.<br>CHECKED | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

## Appendix A

# SELECTION OF HYDROLOGIC MODELS

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## A.1 INTRODUCTION

Many computer models have been developed over the past decade for simulating runoff. This appendix describes the procedures by which the study team selected three models for testing on the Boyne River Pilot Project from the numerous models which are available. Consideration of models was limited to those models which had been developed and or applied in North America. Furthermore, models had to be complete and intact, that is, model development by the study team was ruled out. A further goal was to select models of varying complexity in order to provide an indication of forecast accuracy versus model cost.

## A.2 LITERATURE REVIEW OF HYDROLOGIC MODELS

A total of 18 hydrologic models were given a cursory examination by way of a literature review. This list of models is shown in Table 1 of the Main Report. The report "Analysis of Existing Hydrologic Models, Red River of the North Drainage Basin-North Dakota and Minnesota" (CH2M HILL, 1979) prepared for the U.S. Corps of Engineers in Minneapolis was found helpful in the evaluation of some of the models.

## A.3 MODEL SELECTION CRITERIA

Model selection criteria were prepared for use as an aid in selecting models for detailed review. These criteria were developed based on Manitoba's climatic and physiographic conditions, the availability of data, and the study team's general understanding of simulation requirements. The resulting selection criteria are listed in Table 2 of the Main Report.

The selection criteria were divided into two categories; Primary Model Characteristics and Secondary Model Characteristics. The primary characteristics were considered to be the minimum requirements for the complex models. These requirements were relaxed as necessary, particularly for the simpler models.

Section A of Table 2 lists the model structure criteria. A physically based model was specified, since by its nature such a model was expected to work better in a wider range of conditions than a statistical or parametric model. A continuous model with a variable time step was specified to provide the modellers more scope in optimizing the model parameters. The FORTRAN language was specified because it is the language most familiar to the study team.

Section B deals with data management. The criteria were set to ensure that the model would be designed to handle a large volume of input data efficiently, and that types of data which were not readily available would not be required.

The soil moisture infiltration and snowmelt parameters listed in Section C, D, and E of Table 2 were selected to ensure that the models would be able to respond to conditions which are particularly important in southern Manitoba.

The water budget and conveyance criteria given in Sections F and G reinforce the requirement for a model which simulates the important physical processes.

The complexity criteria in Section H were selected to ensure that the model selected is not too complex to be of practical use.

The secondary model characteristics included in Table 2 contain other aspects which were considered to be desirable, but not essential. These secondary requirements were given less weight than the primary requirements during the model selection process.

In summary, it was deemed that the selected models should be continuous-in-time, as physically based as possible and relatively easy to apply. The criteria were used as a guide only. Models were not rejected if they did not comply with all of the criteria.

## A.4 SELECTION OF MODELS FOR THE PILOT PROJECT

Of the 18 models briefly reviewed, eight were selected for a somewhat more detailed review. These are identified by the asterisks on Table 1. The review of these models is summarized in Table A-1. Based on this review the NWSRFS (Hydrologic Research Laboratory, 1972) and HSPF (U.S. Environmental Protection Agency, 1981) models were originally chosen for detailed testing. These two models were selected because they came closest to meeting all of the primary selection criteria. The relatively simple SLURP model (G. W. Kite, 1975) was also chosen even though its generally parametric approach did not meet selection criteria A(1). It was selected to evaluate how a simple, inexpensive model would perform relative to the other models. The Manitoba API model (MANAPI) is the procedure currently used in Manitoba for forecasting runoff. It does not meet many of the selection criteria.

The SSARR model (U.S. Army Engineering Division, 1975) was initially rejected because its frequent use of index parameters violated criterion A(1) and because it was developed for and has been generally applied in mountainous areas. The SASK 6 model (Water Survey of Canada, 1974) was rejected because its strengths seemed to lie in mountain hydrology. Also its use of fixed parameters to synthesize the streamflow hydrograph from the daily water balances violated criteria A(1) and G(1), and its lack of a surface storage reservoir failed criterion F(1). The QUFM model (Watt et al, 1981) was rejected because its relatively weak deterministic component violated criterion A(1) and because of its emphasis on short term forecasting (one to two days). The MANTHORN model was rejected because a portion of the runoff forecasting technique is statistically-based thereby failing to meet criterion A(1) and because the snowmelt runoff module is untested.

The NWSRFS model was deleted after several months' work because of its great complexity and its relatively poor and incomplete documentation. Also both NWSRFS and HSPF are Stanford-based models and, therefore, have many similarities which could result in a somewhat duplicative effort. The SSARR model was selected to replace the NWSRFS model in spite of its shortfalls listed previously. The SSARR model is less complex and has been widely used with considerable success, although largely in mountainous areas. It is well documented and relatively easy to apply.

The hydrologic models finally selected for testing on the Boyne River Pilot Project thus included the complex HSPF model, the less complex SSARR model and the simple SLURP model. The present MANAPI forecasting procedure, which consists mainly of a statistical-unit hydrograph approach, was selected for use as a bench mark in order to enable an assessment of the accuracy of the other models.

Each of the models selected for the Pilot Project is described briefly in Appendix B, "Basic Procedures of the HSPF, SSARR, SLURP, and MANAPI Hydrologic Models".

## A.5 LITERATURE CITED

1. CH2M HILL, 1979. Analysis of Existing Hydrologic Models, Red River of the North Drainage Basin, North Dakota and Minnesota. CH2M HILL Project No. L13200.A0

2. Hydrologic Research Laboratory, 1972. National Weather Service River Forecast System Forecast Procedures. NOAA Technical Memorandum NWS HYDRO-14, U.S. Department of Commerce.

3. Kite, G.W., 1975. Development of a Hydrologic Model for a Canadian Watershed. Water Resources Branch, Department of the Environment, Ottawa, Ontario.

4. U.S. Army Engineer Division, 1975. Program Description and User Manual for SSARR model. U.S. Army Engineer Division, North Pacific Region, Portland, Oregon.

5. U.S. Environmental Protection Agency, 1981. Hydrologic Simulation Program - FORTRAN, User Manual for Release 8.0, Athens, Georgia.

6. Water Survey of Canada, 1974. General Description of the Saskatchewan River Watershed Model. Technical Report to the PPWB Committee on Hydrology, Calgary, Alberta.

7. Watt, W.E. and Nozdryn-Plotnicki, M.J. Real-Time Forecasting for Flood Damage Reduction, 1981. Proceedings of the International Symposium on Real-Time Operation of Hydro Systems, pp. 741-760, University of Waterloo, Waterloo, Ontario.

## TABLE A-1

## MODEL EVALUATION MATRIX

|  | MODEL<br>CHARACTERISTICS | NWSRFS | HSPF | SSARR-4 |
|---|---|---|---|---|
| BASIC MODEL STRUCTURE | 1. CLASSIFICATION |  |  |  |
|  | a) Type of Processes | physically based, some parts empirical | physically based, some parts empirical | mainly empirical, some index |
|  | b) Continuous or Event | continuous | continuous | continuous |
|  | 2. PROGRAM LANGUAGE | Fortran | Fortran | Fortran |
|  | COMPUTER SYSTEM | IBM main | IBM, CDC main or mini | IBM, CDC main or mini |
|  | 3. COMPUTATIONAL |  |  |  |
|  | a) Annual, Spring, Summer | annual | annual | annual |
|  | b) Time Step | 1,2,3,4,6,8,12,24 hours | minutes to 24 hours | 0.1 to 24 hours |
|  | c) Multiple or Single Watershed | multiple | multiple | multiple |
|  | d) Comments |  |  |  |
|  | 4. DATA REQUIREMENTS<br>Relation to Data Generally Available in Southern Manitoba Watersheds | required data not all available in most watersheds | required data not all available in most watersheds | required data generally available |
|  | 5. APPLICABILITY |  |  |  |
|  | a) Demonstrated Model? | yes | yes | yes |
|  | b) Operational Use? | yes, mainly NWS | limited but gaining | yes, numerous |
|  | c) Model Available? | yes | yes | yes |
|  | 6. DOCUMENTATION LEVEL |  |  |  |
|  | a) Model Processes | fair | good | fair to good |
|  | b) Model Application | poor, very difficult to follow | fair | good |
| DATA MANAGEMENT | 7. COMPLEXITY OF DATA PREPARATION FOR MODEL | moderate | moderate to complex | easy to moderate |
|  | 8. TIME SERIES DATA MANAGEMENT |  |  |  |
|  | a) Complexity | extremely complex | extremely complex | moderate |
|  | b) Flexibility | flexible | very flexible | flexible |
|  | c) Efficiency | fair | good | good |
|  | 9. EASE OF REAL-TIME DATA INPUT | fair | fair | fair to good |

| SASK6 | CWRB (SLURP) | MANAPI | MANTHORN | QUFM |
|---|---|---|---|---|
| mainly empirical, some index | mainly empirical, some index and some physically based | mainly index, some empirical | empirical and index | index and stochastic |
| continuous | continuous | event | continuous | continuous |
| Fortran | Fortran | Manual and Fortran | Fortran | hand calculator |
| CDC | CDC CYBER 74 or IBM | IBM | IBM | N/A |
| annual | annual | spring only | spring and summer | spring only |
| daily | daily | N/A | daily | daily |
| multiple | single | single | single | single |
| | | | | |
| required data generally available | required data generally available | required data generally available | required data generally available | required data generally available |
| yes | yes | yes | yes for rainfall<br>no for snowmelt | yes |
| very limited | no known users | yes, numerous | yes, Manitoba | no known users |
| yes | yes | yes | yes | yes |
| fair to good | fair to good | good | fair to good | fair to good |
| fair to good | good | fair to good | fair to good | fair |
| easy to moderate | easy | very easy | easy | easy |
| moderate | easy | N/A (lumped model) | easy | easy |
| flexible | little flexibility (not required) | N/A | little flexibility ( not required) | little flexibility (not required) |
| good | good | N/A | good | good |
| fair to good | good | good | good | good |

Table A-1 cont'd...

| | MODEL CHARACTERISTICS | NWSRFS | HSPF | SSARR-4 |
|---|---|---|---|---|
| SUMMER SOIL MOISTURE BUDGET | 10. COMPUTATIONAL TIME STEP | variable 6 hours mostly used | 1 to 24 hours | 3 to 24 hours |
| | 11. METHODOLOGY | | | |
| | a) Water Budget or Index | water budget | water budget | water budget |
| | b) Potential Evapotranspiration (PE) obtained from: | pan data, or long-term mean computed PE based on temp. | pan data, adjusted by factor | pan data or some externally computer PE |
| | c) Actual Evapotranspiration (AE) computed from | PE multiplied by a constant depending on vegetation type and soil moisture | PE and soil moisture | long-term average PE and rainfall amount, or pan evaporation and soil moisture |
| | d) Number of Active Storage Reservoirs | four | five | four (three are implicit in routing surface, subsurface and base flow) |
| | e) Losses to Runoff and Deep Percolation Included? | runoff only | yes | runoff only |
| | f) Index Method of Estimating Soil Moisture | N/A | N/A | N/A |
| SNOW COVER | 12. TYPE OF DATA USED | precipitation | precipitation | precipitation |
| | 13. WINTER MELT CONSIDERED? (If precipitation data is used) | yes | yes | yes |
| | 14. EVAPORATION - SUBLIMATION MODELLED? | no | yes | no |
| SNOW MELT | 15. COMPUTATIONAL TIME STEP | 6 hours | 1 to 24 hours | 3,6,12 or 24 hours |
| | 16. METHOD USED | | | |
| | a) Index - method used | | | temperature above base value, times melt factor |
| | b) Energy Budget - parameters used | Ambient temp.<br>Precipitation<br>Evaluation | Ambient temp.<br>Dew pt. temp.<br>Solar radiation<br>Wind speed<br>Precipitation<br>Snow albedo<br>Cloud cover | Ambient temp.<br>Dew pt. temp.<br>Solar Radiation<br>Wind speed<br>Precipitation<br>Snow albedo |
| | 17. ENERGY BUDGET PROCEDURE | | | |
| | a) Sophistication Level | very simplified | complex | quite complex |
| | b) Albedo Modelled? | no | yes | no, but input |

X

| SASK6 | CWRB<br>(SLURP) | MANAPI | MANTHORN | QUFM |
|---|---|---|---|---|
| daily | daily | N/A<br>(lumped model) | daily | daily |
| water budget (HSMB) | water budget | index | water budget and index | index |
| max. temp.<br>min. temp.<br>solar radiation | pan data or some externally computed PE | not input, implicitly handled through weights on monthly precip. | temperature, mean daily sunshine hours, latitude | not input, implicitly handled through recession in API calculation |
| PE and soil moisture, vegetation state | PE is used directly in water balance | not computed | PE and soil moisture | not computed |
| six | two | N/A | two<br>(upper is implicit) | N/A |
|  | runoff only | no | no | no |
| N/A | N/A | API (cumulative weighted monthly precip.) | exponential decay function for monthly precip. | exponential decay function with daily precip. |
| precipitation | precipitation | precipitation | precipitation | precipitation |
| yes | yes | no | no | no |
| no | no | no | no | no |
| daily | daily | N/A | daily | daily |
| temperature above base value, times melt factor | temperature above base value, times melt factor | average no. of degree-days per day during melt period | temperature above base value, times melt factor | temperature above base value, times melt factor |
|  |  |  |  |  |
|  |  |  |  |  |
|  |  |  |  |  |

Table A-1 cont'd...

| | CHARACTERISTICS / MODEL | NWSRFS | HSPF | SSARR-4 |
|---|---|---|---|---|
| | c) Areal Snow Depletion due to Snowmelt Considered? | yes | yes | yes |
| | d) Comments | Assumed values used for many input parameters | | not recommended by authors unless data base is very good |
| WATER BUDGET DURING SPRING | 18. COMPUTATIONAL TIME STEP | variable, 6 hours mainly used | minutes to daily | 0.1 to 24 hours |
| | 19. EVAPORATION, SUBLIMITATION CONDENSATION MODELLED? | evaporation from meltwater only | all when snow on ground. Only evap. when snow gone | average evap. or adjusted pan evap. from meltwater only |
| | 20. SURFACE DEPRESSION STORAGE<br>a) Modelled? | not directly | yes | not directly |
| | b) Method Used | might be handled implicitly with upper layer free water | a storage area is specifically assigned to this | implicit method might work, but no evap. from depression storage |
| | 21. INFILTRATION RATE<br>a) Modelled? | yes | yes | implicitly |
| | b) Method of Modelling | through process parameters, based on moisture in various soil zones, soil type, rate of input | through process parameters, based on soil moisture, permeability slope, rainfall rate | through graphical relationships, based on soil and subsoil moisture |
| | c) Effect of Frozen Soil Modelled? Method | no, but might be handled implicitly by adjusting parameters | infiltration rate can be adjusted based on frost, but method is qualitative | no, but might be handled implicitly by adjusting runoff curve |
| | 22. GROUNDWATER INTERACTION | modelled and base flow computed | modelled and base flow computed | empirical, base flow computed |
| | 23. RUNOFF CALCULATION<br>Method | process parameters using input rate, losses, storages and water transfer rates | process parameters using input rate, losses, storages and water transfer rates | graphical relationships of percent runoff vs soil moisture and subsoil storage |
| | 24. OVERLAND FLOW<br>Method | unit hodrograph | physically-based, using roughness, slope, length of path, etc. | incremental storage routing |

| SASK6 | CWRB<br>(SLURP) | MANAPI | MANTHORN | QUFM |
|---|---|---|---|---|
| | | | | |
| | | | | |
| daily | daily | daily | daily | daily |
| evap. after snowmelt, based on pan data | evaporation must be input | no | no | no |
| not directly | not directly | indirectly | indirectly | no |
| mightly be handled implicitly with upper soil layer | parametric, using a surface storage zone | positive exponent on API variable | positive exponent on soil moisture and API | N/A |
| implicitly | implicitly | no | no | no |
| through process parameters, based on soil moisture in soil zones, soil type | through process parameters, based on moisture in the upper soil zone | N/A | N/A | N/A |
| no, but might be handled implicitly | no, but might be handled implicitly | not usually. Sometimes handled implicitly with a winter temp. index variable | no | no |
| empirical, base flow computed | modelled and base flow computed | not modelled | not modelled | not modelled |
| empirical relationships of input versus soil moisture | specified percent of residual storage in surface and subsurface reservoirs | regression equations and graphical relationships of precip. vs API, melt index | regression equations | graphical or regression equations, based on daily melt and API |
| Nash-type unit hydrograph | surface runoff lagged by n days | not modelled. Lumped into main unit hydrograph | not modelled. Lumped into main unit hydrograph | handled by a transfer function noise model with deterministic and stochastic components |

Table A-1 cont'd...

| METHOD<br>CHARACTERISTICS | | NWSRFS | HSPF | SSARR-4 |
|---|---|---|---|---|
| CONVEYANCE | 25. INTERFLOW<br>Method | unit hydrograph | process parameters obtained by trial calibrations | incremental storage routing |
| | 26. CHANNEL ROUTING<br>Method | Lag and K or Muskingum or Dynamic | Storage type Routing | series of linear reservoirs |
| COMPLEXITY | 27. COST OF MOBILIZATION | very high | high | moderate |
| | 28. COST OF CALIBRATION | high | high | moderate to high |
| | 29. COST OF OPERATIONAL USE | high | high | moderate |
| PROBABLE ACCURACY | 30. BASED ON CALIBRATION DATA AVAILABLE | somewhat better than SASK6 | little information available | performs well in mountain areas |
| | 31. COMMENTS | difficult to calibrate in prairies | | dependent on experience |

| SASK6 | CWRB (SLURP) | MANAPI | MANTHORN | QUFM |
|---|---|---|---|---|
| Nash-type unit hydrograph | lagged by n days | not modelled. Part of unit hydrograph | not modelled. Part of unit hydrograph | same as no. 24 |
| series of linear reservoirs | no built-in routing | Muskingum | no built-in routing | same as no. 24 |
| moderate | low | very low | low | low |
| moderate to high | low | low | low | low to moderate |
| moderate | low | very low | low | low |
|  | as good as SASK6 | fairly good for single-event type snowmelt | fairly good for rainfall runoff | good for one to two days lead time |
|  |  | poor for inter-rupted melt |  | poor past second day |

NOTE: N/A - NOT APPLICABLE

## Appendix B

# BASIC PROCEDURES OF THE HSPF, SSARR, SLURP AND MANAPI HYDROLOGIC MODELS

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## B.1 INTRODUCTION

This appendix contains a brief description of the hydrologic models tested under the Boyne River Pilot Project of the Canada-Manitoba Flood Forecasting Agreement. The models described are the Hydrologic Simulation Program-Fortran (HSPF), The Streamflow Synthesis and Reservoir Regulation (SSARR), the Simple Linear Unit Reservoir Parametric (SLURP), and the Manitoba Antecedent Precipitation Index (MANAPI). The reasons behind the choice of these particular models for detailed testing have been described in Appendix A.

It is not within the scope of this report to describe these models in detail. The intention is simply to provide a general indication of the nature of these models and of the basic procedures which they employ for simulating river flows. The important areas of snowmelt simulation, runoff simulation (lands phase) and conveyance of runoff are addressed.

Detailed information on the HSPF model is contained in a published users manual for this model (United States Environmental Protection Agency (EPA), Athens, Georgia, 30613, July, 1981).

Detailed information on the SSARR model is contained in a program description and user manual for this model (U.S. Army Engineering Division, North Pacific, Portland Oregon, 1975).

Information on the SLURP and MANAPI models is available from the Water Resources Branch, Manitoba Department of Natural Resources, Winnipeg, Manitoba.

## B.2 HSPF MODEL

Hydrologic Simulation Program Fortran (HSPF) is a relatively new software package, Version 5 having been released in April, 1980. However, the hydrologic algorithms used in the model date back to the Stanford IV Watershed model (Crawford and Linsley, 1966). The snowmelt portion uses an energy balance approach. HSPF combines the Stanford and the snowmelt models with river routing and water quality into a convenient package of modules. Additional modules are provided to analyze the data and for ease of presentation and implementation. The version of the model used in this study does not include the water quality component.

Complex hydrologic models usually require large amounts of data, making data preparation and entry a difficult task. In the HSPF model the data input and the transferring of data from one model step to another is simplified through a data handling module known as Time Series Store. Data can be input using the HSPF specified format or a user defined format. Data that is required by the model can come from either an external sequential file or an internal random access file.

### B.2.1 Snowmelt

The general processes involved in snow accumulation and snowmelt are shown on Figure B-1. The algorithms that describe these processes were derived by the Corps of Engineers (1956), Anderson and Crawford (1964) and Anderson (1968). The snowmelt algorithms are generally based on the laws of physics but empirical relationships are used where the physical relationships are now well known.

Five meteorologic time series (data files) are required as input for snowmelt simulation. These are ambient temperature, dew point temperature, solar radiation, wind speed and precipitation. Using these input files and other input parameters such as the water content and the albedo of the snowpack, snowmelt is computed for each of the following sources of heat energy:

1. Net radiation heat, both long wave and solar (RADHT)
2. Convection of sensible heat from the air (CONVHT)
3. Latent heat from condensation of water vapour on the snowpack (CONDHT)
4. Sensible heat from raindrops (RNSHT)
5. Conduction of heat from underlying ground (GMELTR)

All sources of heat are considered positive (incoming to the pack) or zero, except RADHT which can also be negative, i.e. leaving the pack.

Net incoming heat from the atmosphere (the sum of RADHT, CONVHT, CONDHT and RNSHT) is used to warm the snowpack. Any excess heat above that required to warm the snowpack to 0°C is used to melt the pack. Alternately, a net loss of heat will reduce the heat storage of the snowpack, resulting in a cooling of the pack. If the pack cools to a temperature below 0°C, a negative heat storage is computed.

Typically, melt from the ground heat, long wave radiation melt and melt from heat in raindrops are small relative to the other melt factors. The three main energy components producing snowmelt in the Prairies are shortwave (solar) radiation, sensible heat transfer from the air by convection and, occasionally, latent heat of condensation (Gray, 1970). The processes are briefly described in the following subsections.

#### B.2.1.1 Shortwave Radiation Melt

The melt produced by shortwave radiation is a function of the incoming solar radiation (SOLRAD), the reflectance of the snowpack (ALBEDO) and the shading of the snowpack by vegetation (SHADE) as expressed by the following equation:

$$SHORTWAVEMELT = \frac{SOLRAD}{203.2} * (1.0 - ALBEDO)(1.0 - SHADE)$$

SHORTWAVEMELT is in inches per interval when SOLRAD is in langleys per interval. ALBEDO is calculated internally based on the following empirical relationship developed by the Corps of Engineers:

$$\begin{aligned} WINTER: ALBEDO &= 0.85 - 0.07 * (DULL/24.0)^{0.5} \\ SPRING: ALBEDO &= 0.80 - 0.10 * (DULL/24.0)^{0.5} \end{aligned}$$

The dullness of the snowpack (DULL) is an empirical index which provides an indication of the snow's reflectance. It increases by 24 each snow-free day and is decreased by 1000 times the snowstorm water content in inches each day that it snows. DULL ranges from zero for very fresh snow to a maximum of 800 for very old snow. ALBEDO is limited to a minimum value of 0.60 during the winter period when average daily temperatures are below freezing. SHADE is a decimal fraction ranging from 0.0 to 1.0.

#### B.2.1.2 Convection Melt

Convection melt (CONVHT) occurs due to the turbulent heat exchange with the atmosphere and occurs when the air temperatures is above 0°C Convection melt is calculated according to the following empirical relationship:

$$\begin{aligned} CONVHT &= (AIRTEMP - 32.0) * (1.0 - 0.3 * \frac{MELEV}{10000.0}) * \\ &\quad CCFACT * 0.00026 * WINMOV \end{aligned}$$

where: CONVHT = melt in inches due to convection
AIRTMP = air temperature (°F)
MELEV = mean elevation in feet
CCFACT = parameter to adjust melt factor to suit observed melt
WINMOV = wind in miles/time interval

#### B.2.1.3 Condensation Melt

Condensation Melt (CONDHT) is important when a warm moist air mass presides over the snowpack. Condensation onto the snowpack occurs when the air has a vapour pressure greater than 6.108 millibars, i.e., when the dew point is above 0°C. Condensation melt is computed according to the following formula:

$$CONDHT = 8.59(VAP - 6.108) * CCFACT * 0.00026 * WINMOV$$

where: CONDHT = melt in inches due to condensation
VAP = air vapour pressure in millibars
CCFACT = parameter to correct melt factor to suit observed melt
WINMOV = wind in miles/time interval

### B.2.2 Runoff Calculation Procedures

The lands phase processes are simulated in the PWATER section of the HSPF Model. The process modelled are illustrated in Figure B-2. The algorithms simulating these land related processes are based on the original research for the LANDS Subprogram of the Stanford IV Watershed Model (Crawford and Linsley 1966). This lands component has been incorporated into many models and has been used successfully to simulate the hydrologic response of widely varying watersheds.

Moisture is input to the system via snowmelt and rainfall. Moisture is lost through evapotranspiration. Some of the precipitation that falls on a land segment is intercepted by vegetation and evaporates. Water applied to the land surface is subjected to an infiltration decision. Water that does not immediately infiltrate goes into upper zone storage. From there it can run off as overland flow or interflow, or temporarily remain in upper zone storage. Upper zone storage represents water stored in shallow surface depressions and in the upper root zone.

Water that does not run off from the upper zone will eventually either evaporate or percolate down to the lower zone. The lower zone storage governs the infiltration rate of water into the soil. As the lower zone fills the infiltration rate decreases. Water is passed from the lower zone to groundwater, based on the moisture storage in the lower zone relative to its nominal storage. This nominal storage is approximately 80 percent of the lower zone storage capacity. Outflow from groundwater storage constitutes baseflow.

The actual evapotranspiration (ET) is computed from the potential evapotranspiration (PET), an input time series, and the moisture supply in the watershed in the following manner. An attempt is made to meet the PET demand from various sources, on a priority basis. First ET occurs from vegetation along the stream channel. The rate at which this occurs is dependent on the baseflow into the stream. Secondly, ET occurs from interception storage, based on free evaporation. Thirdly, ET occurs from the upper zone, depending on the amount of water in storage. Fourthly, if ET is not yet satisfied, it occurs from groundwater based on a percentage of baseflow. Lastly, ET occurs from the lower zone. ET from the lower zone is dependent on the amount of water in storage relative to its nominal storage and on the remaining PET demand. The latter PET demand is modified by monthly factors to reflect the state of vegetation, as described in Appendix D under the section on evapotranspiration.

Infiltration is the key component that controls watershed response. Water runs off faster as overland flow than as baseflow. Therefore, the ratio of peak flow to runoff volume is much greater where infiltration is slow and overland flow is relatively high. The infiltration capacity is the maximum rate at which soil will accept water and is a function of both the fixed and variable characteristics of the watershed. The fixed characteristics consist primarily of soil permeability and land slope, while variable characteristics are the soil surface conditions, rainfall intensity and soil moisture content.

The complex process of infiltration is approximated by two interlocking calculations. For each time step these calculations estimate the direct infiltration into the soil profile and the increase in soil moisture storage that results in delayed infiltration. The relationship between interflow, overland flow and infiltration is depicted on Figure B-3. The locations of Line I and II on this Figure are calculated as follows:

$$\underline{LineI} : IBAR = (INFILT/LZS/LZSN)^{INFEXP} * INFFAC$$

where: IBAR = midpoint of Line I
INFILT = infiltration parameter
LZS = actual lower zone storage
LZSN = nominal lower zone storage
INFEXP = infiltration exponent
INFFAC = frozen ground factor

$$\underline{LineII} : IIBAR = IBAR * INTFW * 2.0^{(LZS/LZSN)}$$

where: IIBAR = midpoint of Line II
INTFW = interflow inflow parameter

The slope of the line which accounts for the areal variation of the infiltration rate throughout the segment is defined by

$$IMAX = INFILD * IBAR$$

These equations allow infiltration to vary dynamically with the moisture supply (MSUPY). Given a constant moisture supply over time the lines will lower, resulting in less infiltration and more water going to overland flow and interflow. An increase in the intensity of rainfall results in more water going into potential direct runoff.

### B.2.3 Conveyance

Conveyance is the process of converting runoff depths or volumes into a streamflow hydrograph. This involves simulating the movement of runoff from the land surface to a stream channel and, for larger watersheds, the channel routing of the water to the outlet of the watershed.

Outflow from the watershed occurs in three ways; overland flow, interflow and groundwater flow. Each is handled differently with respect to conveyance. These simulations are performed in the PWATER section of the HSPF model.

Overland flow is treated as an unsteady flow problem. It is solved by using the empirical Chezy-Manning equation (Crawford and Linsley, 1966) that relates outflow depth to detention storage. Overland flow conveyance is based on the surface roughness, the length of the overland flow plane and slope of the land segment.

Interflow is routed through the soil in the same way streamflow would be routed through an uncontrolled lake. As inflow increases interflow storage increases, thereby increasing the outflow. The relationship between increase in storage and increase in outflow is controlled by an interflow parameter which is similar to an outlet capacity curve for a lake. Outflow from interflow storage is input to the nearest stream channel.

Outflow from groundwater storage is also proportional to the volume of water in groundwater storage. Basic outflow is determined by multiplying groundwater storage by a recession constant. A variable outflow rate is added to allow for increased outflow resulting from past active groundwater inflows. Outflow from groundwater storage is input to the nearest stream channel.

Channel routing is treated as a series of reservoir routings. Tables of stage versus discharge and stage versus storage are required as input for each river reach. Local inflows are added at the upstream end of each reach. Further details on stream channel routing are contained in Appendix F.

## B.3 SSARR MODEL

The SSARR model, which was originally designed for mountainous areas, has a number of features which are not pertinent to prairie watersheds. These features relate to the accumulation and melting of snow in regions of dramatic elevation change. The following discussion will be limited to the portions of the SSARR model which pertain to prairie environments only. Figure B-4 depicts the basic watershed processes modelled by the SSARR model.

### B.3.1 Snowmelt

There are two methods in the SSARR model that may be used to calculate the amount of snowmelt. They are the temperature index method and the generalized snowmelt equation. The latter was chosen for use in this study because it is the more rigorous method. The snowmelt equation was originally developed through the Cooperative Snow Investigations conducted in the Northwest United States (Snow Hydrology - Corps of Engineers, U.S. Army, 1956). It is expressed as follows:

$$\begin{aligned} M &= K^1(1-F)*(0.0040I)*(1-a)+K*(0.0084v)*(0.22(T_a-32) \\ &\quad +0.78(T_d-32))+0.029F*(T_a-32) \end{aligned}$$

where: M = Snowmelt rate in inches per day
$T_a$ = Air temperature in degrees Fahrenheit
$T_d$ = Dew point temperature in degrees Fahrenheit
$v$ = Wind speed in miles per hour
I = Global solar radiation on a horizontal surface, in langleys
$a$ = Average snow surface albedo as a decimal fraction
$K^1$ = Basin shortwave radiation melt factor
F = Average forest canopy cover, expressed as a decimal fraction
K = Convection-condensation melt factor as a decimal fraction

The values for K, $K^1$ and F are constants for a basin whereas the values for I,$a$,$v$, $T_a$, and $T_d$ are time dependent data and are entered on a period by period basis.

The first term of this equation represents melt caused by shortwave solar radiation. The following two terms represent convection and condensation melt respectively, while the final term represents melt resulting from long-wave radiation.

The SSARR model contains various options to simulate the depletion of a snowpack on a watershed. However, most of these have been designed for mountainous regions. The option best suited for a prairie environment is the snow band option with the total watershed encompassed in one elevation band (E. M. Davis, pers. comm.). In it the basin is assumed to be either 100 percent snow covered or not covered at all.

### B.3.2 Runoff Calculation Procedures

The simulation of the movement of water through the hydrologic cycle is accomplished, for the most part, by the use of a series of graphs (curves) and index relationships.

Initially water is input to the system as rainfall, snowmelt or a combination of rain and snowmelt. The moisture input then is divided into two components, one portion going to runoff and the other to soil moisture. This division is accomplished by means of a runoff percent curve which is input to the model. An example of such a curve is shown on Figure B-5. The percent runoff is dependent on the wetness of the watershed at the start of the input event, i.e. on the Soil Moisture Index (SMI). When the SMI is extremely high, the runoff coefficient approaches 100 percent.

An initial runoff percent curve is normally drawn from experience and a general knowledge of the watershed. The curve is then adjusted as part of the model calibration procedure.

For each time period modelled, the soil moisture is adjusted by the balance of moisture input and moisture losses to runoff and evapotranspiration during the period.

In equation form this relationship is:

$$SMI2 = SMI1 + (WP - RGP) - (DKE * ETI)$$

where: SMI1 = Soil Moisture Index (in inches) at the beginning of the period
SMI2 = Soil Moisture Index (in inches) at the end of the period
WP = Moisture input (in inches)
RGP = Runoff generated for the period
DKE = Fraction of the potential evapotranspiration which actually occurs from the soil
ETI = Potential evapotranspiration (in inches)

The developers of the SSARR model realized that soils dry out at different rates dependent upon the type of soil, the amount of moisture in the soil and the type of vegetation. The rate of drying is specified by the application of a soil drying curve which relates DKE to SMI. An example of such a curve is given on Figure B-6. The formulation of an appropriate soil drying curve is part of the model calibration procedure.

The total runoff obtained from the runoff percent curve for a given computational time interval is subsequently divided into baseflow and direct runoff. The proportion of the runoff going to baseflow is determined by means of a Baseflow Percent Curve. This curve relates the percent baseflow to a Baseflow Infiltration Index (BII). An example of such a curve is shown on Figure B-7. This index is adjusted for each time period based on a manipulation of its previous value and the moisture input, as follows:

$$BII_2 = BII_1 + (24 * RG - BII_1) * \frac{PH}{TSBII + PH/2}$$

where: $BII_1$ = Baseflow Infiltration Index (in inches per 24 hours) at the beginning of the period
$BII_2$ = Baseflow Infiltration Index (in inches per 24 hours) at the end of the period
RG = Runoff rate in inches per hour (i.e. RGP/PH)
PH = Period length in hours
TSBII = Time delay or time of storage for calculation of the change in BII. If desired, separate time of storage values may be designated for rising and falling limbs of the hydrograph.

**The Baseflow Percent Curve quantifies the concept that when there is, or has recently been, a high moisture input, baseflow will be a small portion of the total runoff. For a wet basin, increasing proportions of further moisture inputs will go to direct runoff.**

Another option allows the modeller to input a baseflow input limit which conceptually represents the limits on percolation to groundwater based on the existing amount in baseflow storage (represented by the BII).

The direct runoff, obtained as the residual of total runoff minus baseflow, is further subdivided into subsurface (interflow) and surface runoff by means of a curve relating direct runoff to surface runoff. This is known as a Surface/Subsurface Runoff Curve. An example is shown on Figure B-8. The direct runoff rate in inches per hour is found on the abscissa and represents the total surface and subsurface runoff rate. The values along the ordinate axis are the corresponding surface runoff rates in inches per hour.

The dashed line on Figure B-8 is drawn at 45° from the horizontal and is the maximum level at which the curve could be drawn. It represents a situation in which the direct runoff is composed only of surface runoff, i.e. there is no interflow. The solid curve is an example for a particular watershed. The vertical distance between the solid curve and the dashed line represents the input rate to subsurface runoff (interflow), while the vertical distance below the solid curve represents the input rate to surface flow. It should be noted that the ratio of surface to subsurface input is not constant but changes with the total rate of direct runoff. Also the maximum input to subsurface flow is dependent upon the infiltration capacity of the soil. This is represented by the solid curve running parallel to the dashed line above a specified value of the direct runoff rate. Any direct runoff inputs in excess of the infiltration capacity of the soil become surface runoff.

### B.3.3 Conveyance

The volumes of runoff assigned to the components of baseflow, subsurface flow and surface flow for each model time step are routed separately before they enter the stream. This process accounts for the time of travel and the attenuation of the component flows as they travel above or below the surface toward the stream channel. The routing procedure used is the standard SSARR method of routing. The method may be thought of as a series of identical reservoirs which delay runoff. In applying the SSARR routing method, two parameters need to be chosen: the number of reservoirs (or phases) and the time of storage per phase. The time lag and the attenuation of each component of runoff may be estimated by judiciously choosing these two parameters for each component. This method allows virtually unlimited flexibility in the time distribution of runoff.

## B.4 SLURP MODEL

The Simple Linear Unit Reservoir Parametric (SLURP) model used in this study is considerably modified from that developed by Kite in 1975. Portions of the model were amended and many new features were included in order to make the model more rigorous and complete (A. A. Warkentin, pers. comm.).

The basic processes of the SLURP model are shown on Figure B-9. The SLURP model is almost entirely parametric since most model processes are not physically-based. The model makes use of four moisture retention reservoirs when there is a snowpack and two reservoirs when there is no snowpack. The model runs on a daily time interval. Input data include daily temperature, daily precipitation, and daily potential evapotranspiration. Output consists of daily simulated flows and monthly totals for various components of the water balance.

Calibration of the model requires the choice of appropriate values for 21 model parameters which control the interactions between the model processes and the data and basin conditions. These parameters are listed in Table B-1.

The model contains a parameter optimization option whereby the model can select the "best fit" values for each parameter. Following some trial runs this option was no longer used because of its tendency to select values which were physically unrealistic.

### B.4.1 Snowmelt

Daily snowmelt is computed using the following formula:

$$MELT = (X12) * (T - X13) + PT/79.7$$

where: MELT = daily melt (mm)
X12 = input melt factor (mm/°C)
T = mean daily ambient temperature (°C)
X13 = input snowmelt threshold temperature (°C)
P = rainfall for the day (mm)

It is apparent that this is not an energy budget type of melt computation, although the second term does represent melt from the heat of raindrops. This second term usually contributes little to the snowmelt except when there are large amounts of warm rain. The first term represents mainly convective heat transfer from the air to the snowpack. This term could also represent some radiation melt if X12 and X13 are appropriately chosen, because air temperature and radiation are correlated to some extent.

The melt formula does not directly account for possible snowmelt caused by the latent heat of condensation, which can be considerable when the dew point of the airmass is above 0°C. An attempt was made to remedy this problem by the use of variable values for the melt factor, X12. A relationship was constructed by which the melt factor may be adjusted based on the ambient temperature and the dew point temperature (A. A. Warkentin, pers. comm.), as shown on Figure B-10.

As shown on Figure B-9, the SLURP model removes the computed daily snowmelt from the snowpack and adds it to a SNOWMELT reservoir. On the day following the disappearance of the snowpack, the entire contents of the SNOWMELT reservoir are transferred to the SURFACE reservoir.

### B.4.2 Runoff Calculation Procedure

The SLURP model has a total of four reservoirs known as SNOW, SNOWMELT, SURFACE and GROUNDWATER, representing the snowpack, meltwater, surface water and groundwater respectively. These are shown on Figure B-9. All reservoirs except SNOW contribute to runoff when their respective storages are greater than zero. The output from SNOW is the daily snowmelt which enters the SNOWMELT reservoir. Runoff occurs simultaneously from the SNOWMELT and SURFACE reservoirs only when there is a snowpack. When the snowpack is completely melted, i.e. when the SNOW reservoir is reduced to zero, the contents of the SNOWMELT reservoir are transferred to the SURFACE reservoir.

Snowfall is the only input to the SNOW reservoir. Precipitation is considered to be in the form of snow if the mean daily temperature is below the pre-set rain/snow temperature, X7.

Input to the SNOWMELT reservoir is comprised of rainfall and snowmelt. Outputs are evaporation, percolation and runoff.

Input to the SURFACE reservoir is comprised of rainfall, plus the contents of the SNOWMELT reservoir upon the disappearance of the snowpack. Outputs are evaporation, percolation and runoff.

Inputs to the GROUNDWATER reservoir consist of the percolated water from the SNOWMELT and/or SURFACE reservoirs. Evaporation and runoff are its outputs.

The four storage reservoirs in the SLURP model all operate in the same manner. The storage of each reservoir is updated daily based on a priority system. First the input moisture is added to the existing storage. Evaporation is then subtracted from the storage, and if there is still water remaining in storage, percolation is subtracted. Finally runoff is taken from the reservoir if there is water remaining. This basic operation of the reservoirs is shown on Figure B-11.

The potential evapotranspiration for the SNOWMELT, SURFACE and GROUNDWATER reservoirs may be adjusted in three steps. First the daily potential evapotranspiration (PE) input to the model is multiplied and/or incremented by the adjustment factors X20 and X21 to produce a revised estimate of potential evapotranspiration loss (PEL). Then the resultant PEL value is multiplied by the input factor X6 to produce an estimate of the potential evapotranspiration loss from the SNOWMELT and SURFACE reservoirs. The remaining PEL is then assigned as a potential loss from the

GROUNDWATER reservoir.

Actual evapotranspiration is computed by the model for each operating reservoir from the computed PEL for the respective reservoirs, based on the availability of water in the storage reservoirs. If the reservoir moisture content at the start of a day equals or exceeds the reservoir PEL for that day, the full PEL for the reservoir is substracted from its storage. If the starting reservoir storage is less than PEL, the reservoir storage is simply reduced to zero.

The amounts of water going to runoff and percolation from any particular reservoir for each model time step are fixed proportions of the storages in the reservoir. These proportions must be determined through model calibration and are input via the parameters X8, and X10 for the SURFACE reservoir and X14 and X15 for the SNOWMELT reservoir. The parameter X16 denotes the proportion of the GROUNDWATER reservoir going to runoff each day.

Initial values of storage must be input for all reservoirs except SNOWMELT which always has an initial storage of zero. These are input through the parameters X1, X2 and X3. A simulation run for a snowmelt event should have a starting date in advance of melting temperatures. If there is melt followed by a cold spell, the model will not simulate a re-freezing of the meltwater. If significant re-freezing occurs it is necessary to restart the model during the cold spell with new initial storages.

The moisture storage capacity of the SURFACE storage reservoir is specified using the capacity parameter X9. The parameter may be initially estimated from soil type but, as with other model parameters, must be defined through model calibration. Any excess input which would raise the storage above this capacity is assigned to runoff.

### B.4.3 Conveyance

The daily runoffs from each of the SURFACE, SNOWMELT and GROUNDWATER reservoirs may be lagged by any number of days using the lag parameters X17, X18 and X19. There is no provision in the model for the attenuation of the outflow hydrographs from these reservoirs.

## B.5 MANAPI MODEL

The Manitoba Antecedent Precipitation Index (MANAPI) model (Warkentin, 1973) employs either graphical or statistical methods to predict the runoff depth for a watershed. The unit hydrograph for that watershed, which is a time series of the flows that would result from one inch of runoff, is then multiplied by the runoff depth to obtain predicted daily flows for the watershed. For some basins, peak flows are also provided directly from regression equations. Variations of such basic index procedures have been used by most flow forecasting agencies in Canada in the past and are still used operationally by many agencies in the 1980's.

The MANAPI model is not continuous-in-time with respect to the spring snowmelt runoff since it is a "lumped" model. However the model is continuous-in-time from the date of the snowmelt peak onward, making it possible to predict additional streamflow resulting from spring rainstorms. This rainfall runoff forecasting procedure was calibrated on the Boyne River for purposes of this study but has not yet been applied to other watersheds in Manitoba.

### B.5.1 Snowmelt

The simple degree-day method is used in the MANAPI procedure. For each day the mean daily temperature minus a threshold temperature (which is usually 0°C is computed. The resultant values are accumulated and plotted for the snowmelt period giving a plot of cumulative degree-days versus the date. The average number of degree-days per day during the active snowmelt period, known as the melt index (MI) is computed. The melt index is one of the independent variables used in developing a regression equation to predict runoff volume or peak flow. An example of a melt index calculation is shown on Figure B-12.

### B.5.2 Snowmelt Runoff Forecasting

The following parameters are computed for each year of hydrometric record in the watershed:

1. Spring runoff depth over the watershed (RO).
2. Spring peak flow (PK).
3. Antecedent precipitation index (API) based on a summation of weighted monthly percent-of-normal precipitation from May to October of the previous year using the following weights:

| May | June | July | August | September | October |
|---|---|---|---|---|---|
| .07 | .08 | .12 | .18 | .25 | .30 |

4. Total precipitation (TP) from November 1 of the previous year up to the date of the spring peak flow at the watershed outlet.
5. Melt index (MI) based on the average number of degree-days per day during the active melt period.

The data is then analyzed using either the graphical or statistical methods. The graphical method involves plotting spring runoff depths versus total winter plus effective spring precipitation for all the years of hydrometric record. Each plotted value is labelled with the computed API for that year. A family of API curves is then drawn to best fit the API values on the plot. An example is shown on Figure B-13. Runoff may be predicted by following through the graph with the appropriate values of TP and API.

The statistical method involves multiple regression analysis of the data using runoff depth or peak flow as the dependent variable. An equation of the following form is normally used:

$$RUNOFF = C * (TP)^k * (API)^l * (MI)^m$$

The values of C,$k$, $l$ and $m$ in the equation are computed by Fletcher Optimization (Fletcher, 1965), an optimization procedure built into the regression program. This type of equation allows for non-linearity between the dependent and independent variables and also allows for interaction between the independent variables.

In the forecast mode, runoff depth for a particular spring may be predicted by simply entering the computed parameter values for that event into the derived equation for runoff. The peak flow for a spring snowmelt runoff event may be predicted directly using a regression equation with peak flow as the dependent variable, or it may be predicted by multiplying the forecasted runoff depth by the peak of an appropriate unit hydrograph for the watershed.

When forecasting flows with the MANAPI model, spring rain falling prior to the peak flow from snowmelt is added to the snowpack. Runoff from such rain is thus forecast together with the snowmelt runoff. Runoff characteristics for rain falling after the snowmelt peak are usually quite different due to thawing of the soil and changes in pothole storage, etc. A separate continuous-in-time procedure has therefore been devised to forecast runoff from these later rainstorms. This procedure is described in the following subsection.

### B.5.3 Rainfall Runoff Forecasting

Rainfall runoff forecasting involves firstly the calculation of an initial antecedent moisture condition (IAMC) for the watershed at the time of the peak from snowmelt runoff (A. A. Warkentin, pers. comm.). This is computed using pertinent variables from the spring runoff forecasting procedure and an estimate of the total snowmelt runoff (RO) volume for the year in question. The derived relationship is as follows:

$$IAMC = (0.03 * API + TP - RO)/2$$

A daily antecedent moisture condition (AMC) is then computed for each day following the date of the peak from snowmelt runoff, as follows:

$$\begin{aligned} AMC_1 &= (IAMC + P_1)^{0.95} \\ AMC_2 &= (AMC_1 + P_2)^{0.95} \\ AMC_{n+1} &= (AMC_n + P_{n+1})^{0.95} \end{aligned}$$

where subscripts refer to the number of days after the date of the peak flow from snowmelt runoff.

To prepare a rainfall runoff forecast, the storm rainfall total and the AMC for the day preceding the start of the storm are computed and entered into a graphical rainfall-runoff forecasting procedure for the watershed of the type shown on Figure B-14. The predicted runoff depth is read on the ordinate. Such a relationship is prepared from an analysis of past rainstorms in the watershed.

### B.5.4 Conveyance

With the MANAPI model, conversion of the predicted runoff depths into hydrographs of daily flows at the watershed outlet is effected through the use of unit hydrographs. Unit hydrographs are constructed for moderate and for fast snowmelt rates. The unit hydrographs for fast snowmelt are also normally used for runoff from rainstorms, although separate unit hydrographs may be prepared for specific rainstorm intensities or durations. Unit hydrographs are constructed from previous runoff events.

## B.6 LITERATURE CITED

1. Anderson, E.A. and Crawford, N.H., 1964. The Synthesis of Continuous Snowmelt Runoff Hydrographs on a Digital Computer. Department of Civil Engineering, Stanford University, Stanford, California.

2. Anderson, E.A., 1968. Development and Testing of Snow Pack Energy Balance Equations. Department of Civil Engineering, Stanford University, Stanford, California.

3. Brooks, K.N., Davis, E.M., Kuehl, D.W., 1975. Program Description and User Manual for the SSARR Model - Program 724-K-5-G0010. Engineering Division, U.S. Army, Portland Oregon.

4. Crawford, N.H. and Linsley, R.K., 1966. Digital Simulation in Hydrology: Stanford Watershed Model IV, Technical Report No. 39. Department of Civil Engineering, Stanford University, Stanford, California.

5. Fletcher, R., 1965. Function Evaluation Without Evaluating Derivatives: A Review. Computer Journal, Volume 8, pp. 33-41.

6. Gray, D.M., 1970. Principles of Hydrology. College of Engineering, University of Saskatchewan, Saskatoon, Saskatchewan.

7. Johanson, R.C., Imhoff, J.C., Kittle, J.L., Donigan, A.S. Jr., Davis, H.H. Jr., 1981. User's Manual for Hydrological Simulation Program - Fortran (HSPF), Release 70. U.S. Environmental Protection Agency, Athens, Georgia, 30613.

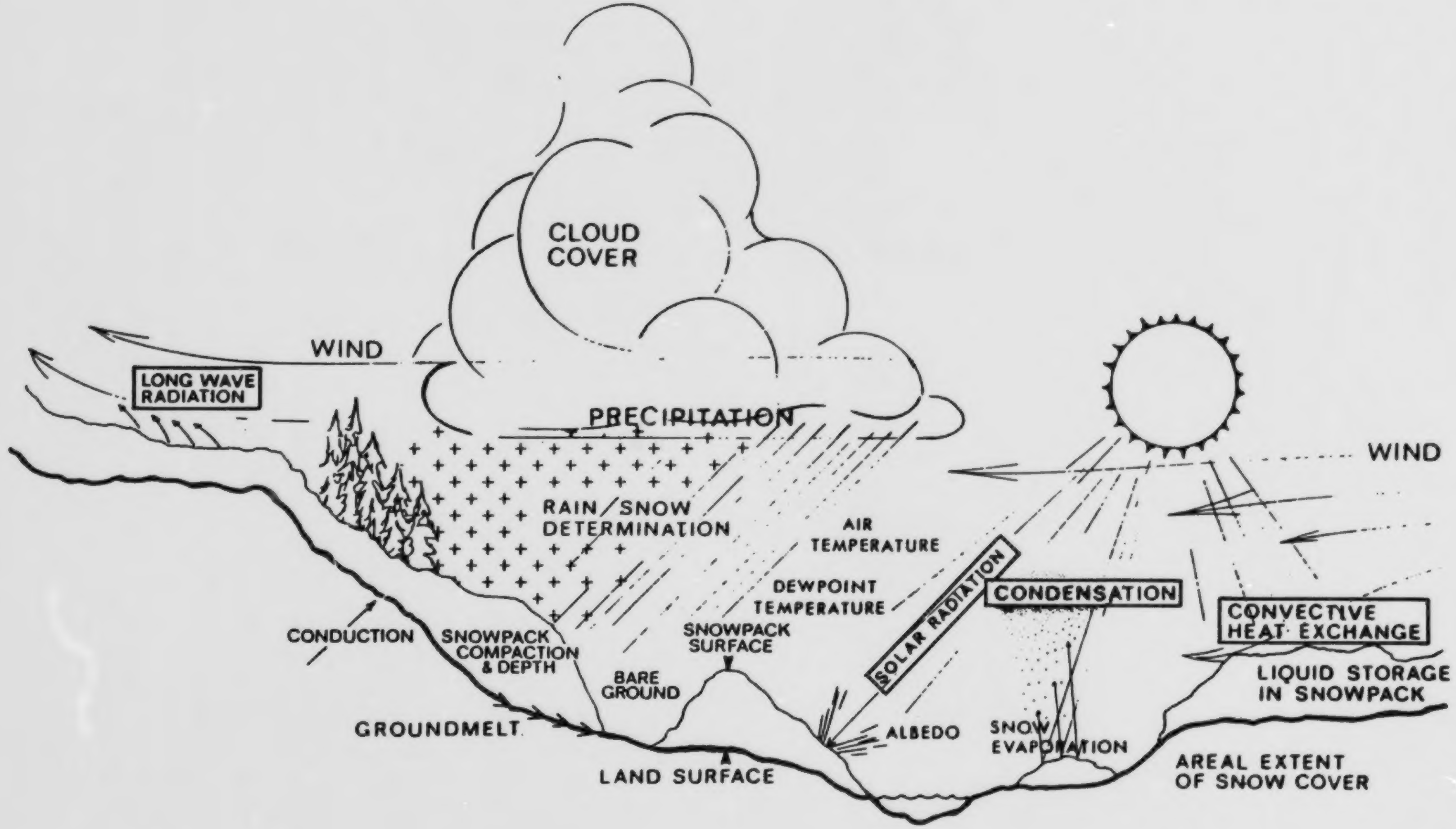
8. Kite, G.W., 1975. Development of a Hydrologic Model for a Canadian Watershed. Water Resources Branch, Department of the Environment, Ottawa, Ontario.

9. Snow Hydrology, 1956. North Pacific Division Corps of Engineers, U.S. Army, Portland, Oregon.

10. Warkentin, A.A., 1973 (unpublished). Runoff Forecasting in the Boyne River Watershed Upstream of Carman, Manitoba.

## Table B-1
## Input Parameters For The Slurp Hydrologic Model

| Parameter Number | Description |
|---|---|
| X1 | Initial Water Content of the Snowpack (mm) |
| X2 | Initial Content of the SURFACE Reservoir (mm) |
| X3 | Initial Content of the GROUNDWATER Reservoir (mm) |
| X4 | Total Potential Evapotranspiration (mm) |
| X5 | Rainfall Adjustment Factor |
| X6 | Proportion of Evapotranspiration from the SURFACE and SNOWMELT Reservoirs |
| X7 | Rain/Snow Division Temperature (°C) |
| X8 | Proportion of SURFACE Reservoir going to Runoff |
| X9 | Capacity of the SURFACE Storage Reservoir (mm) |
| X10 | Proportion of Surface Reservoir going to Percolation |
| X11 | Snowmelt Factor (mm/Celsius degree - day) |
| X12 | Snowmelt Factor for Rain on Snow Condition |
| X13 | Snowmelt Threshold Temperature (°C) |
| X14 | Proportion of SNOWMELT Reservoir going to Runoff |
| X15 | Proportion of SNOWMELT Reservoir going to Percolation |
| X16 | Proportion of GROUNDWATER Reservoir going to Runoff |
| X17 | Lag time for Runoff from SURFACE Reservoir |
| X18 | Lag time for Runoff from SNOWMELT Reservoir |
| X19 | Lag time for Runoff from GROUNDWATER Reservoir |
| X20 | Multiplying Factor for Input Potential Evapotranspiration |
| X21 | Additive Adjustment to Input Potential Evapotranspiration |


CLOUD COVER
WIND
LONG WAVE RADIATION
PRECIPITATION
RAIN/SNOW DETERMINATION
AIR TEMPERATURE
DEWPOINT TEMPERATURE
SOLAR RADIATION
CONDENSATION
WIND
CONVECTIVE HEAT EXCHANGE
LIQUID STORAGE IN SNOWPACK
CONDUCTION
SNOWPACK COMPACTION & DEPTH
BARE GROUND
SNOWPACK SURFACE
ALBEDO
SNOW EVAPORATION
GROUNDMELT
LAND SURFACE
AREAL EXTENT OF SNOW COVER

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## SNOW ACCUMULATION AND MELT PROCESSES-HSPF MODEL

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>G.E.M. | DRAWN<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE | SHEET OF | FILE NO. |

FIGURE B-1

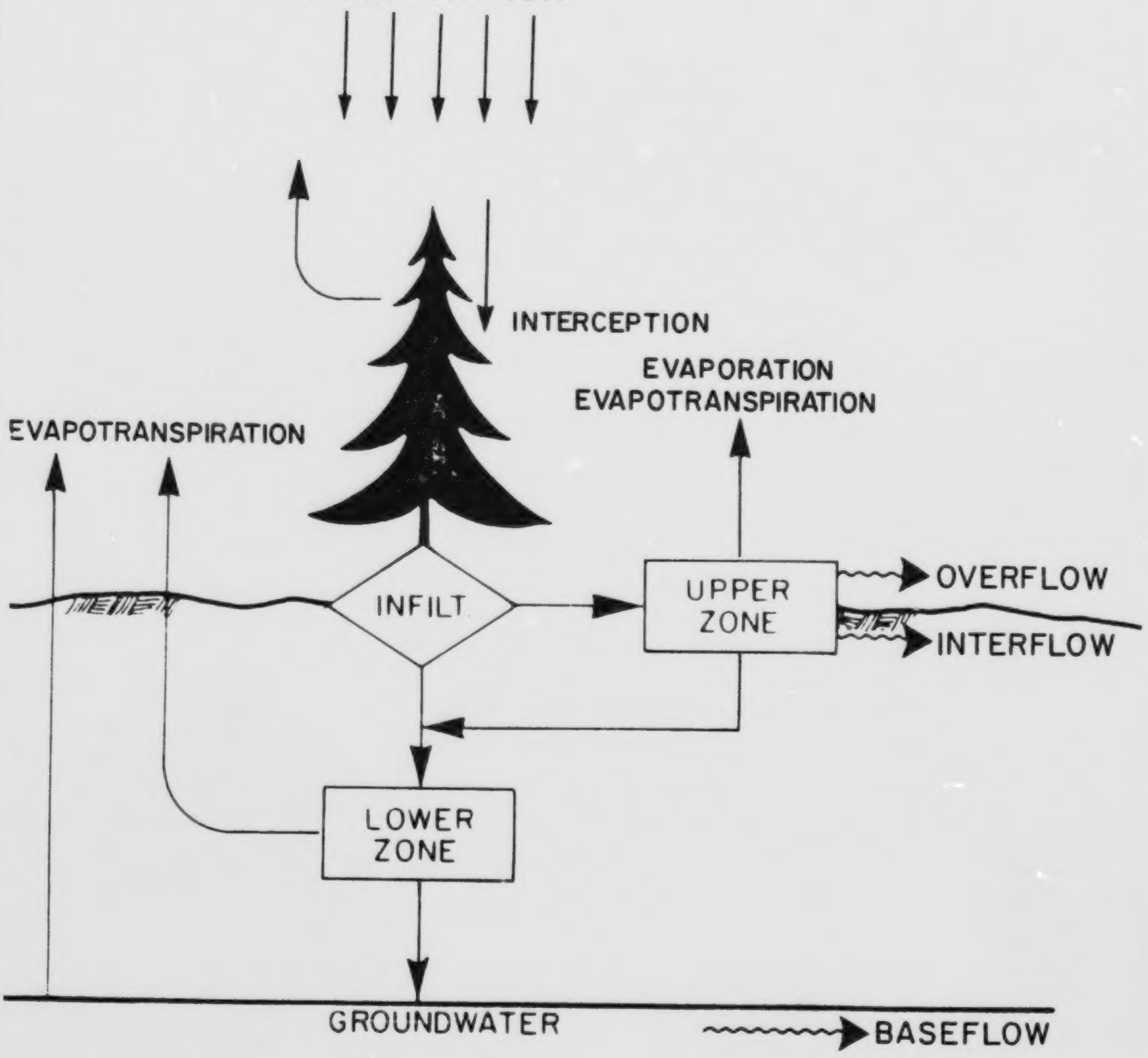

PRECIPITATION
INTERCEPTION
EVAPORATION
EVAPOTRANSPIRATION
EVAPOTRANSPIRATION
INFILT.
UPPER ZONE
OVERFLOW
INTERFLOW
LOWER ZONE
GROUNDWATER
BASEFLOW

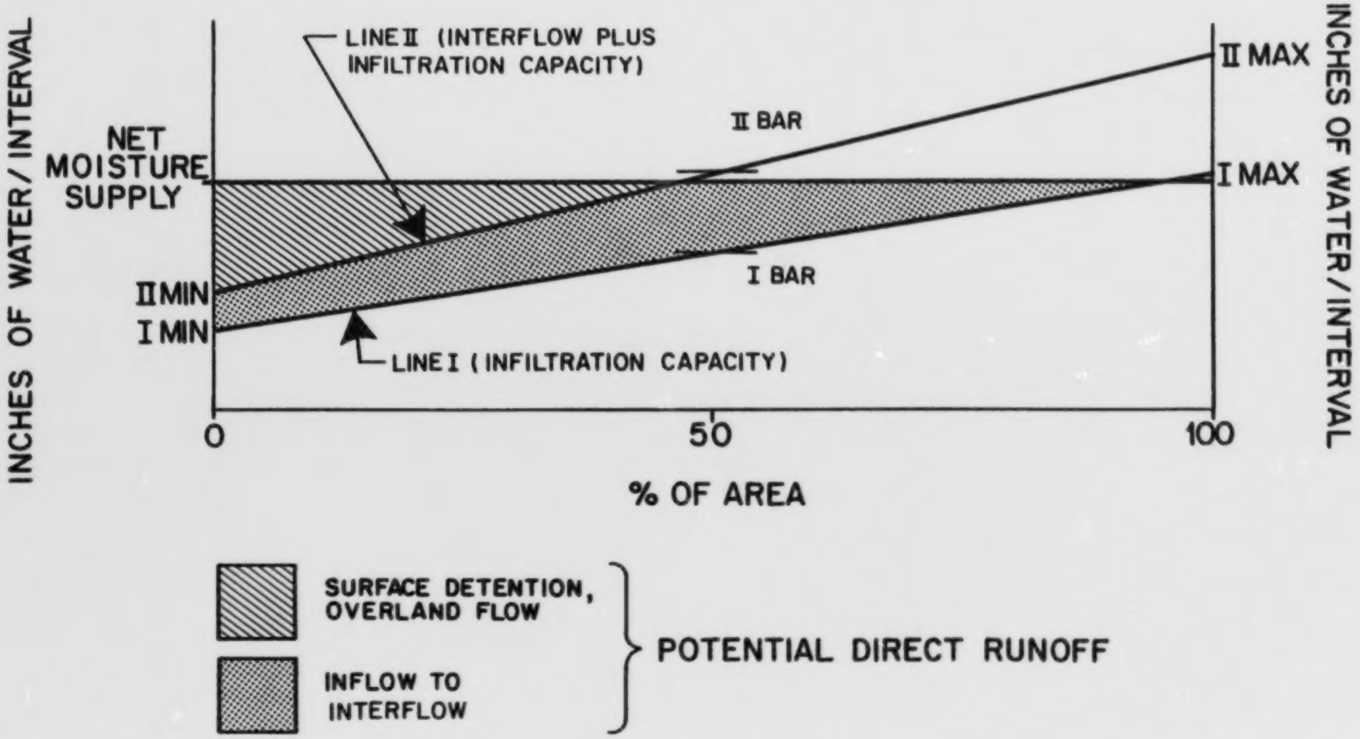
FIGURE B-2

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

# HYDROLOGIC PROCESSES MODELLED BY PWATER OF THE HSPF MODEL

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
**WATER RESOURCES BRANCH**

PREPARED G.E.M.
DRAWN
CHECKED A.W.
SUBMITTED
APPROVED
SCALE AS SHOWN
DATE
SHEET OF
FILE NO.



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| G.E.M. | CHECKED A.W. | | |

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## DETERMINATION OF INFILTRATION, INTERFLOW AND OVERLAND FLOW-HSPF MODEL

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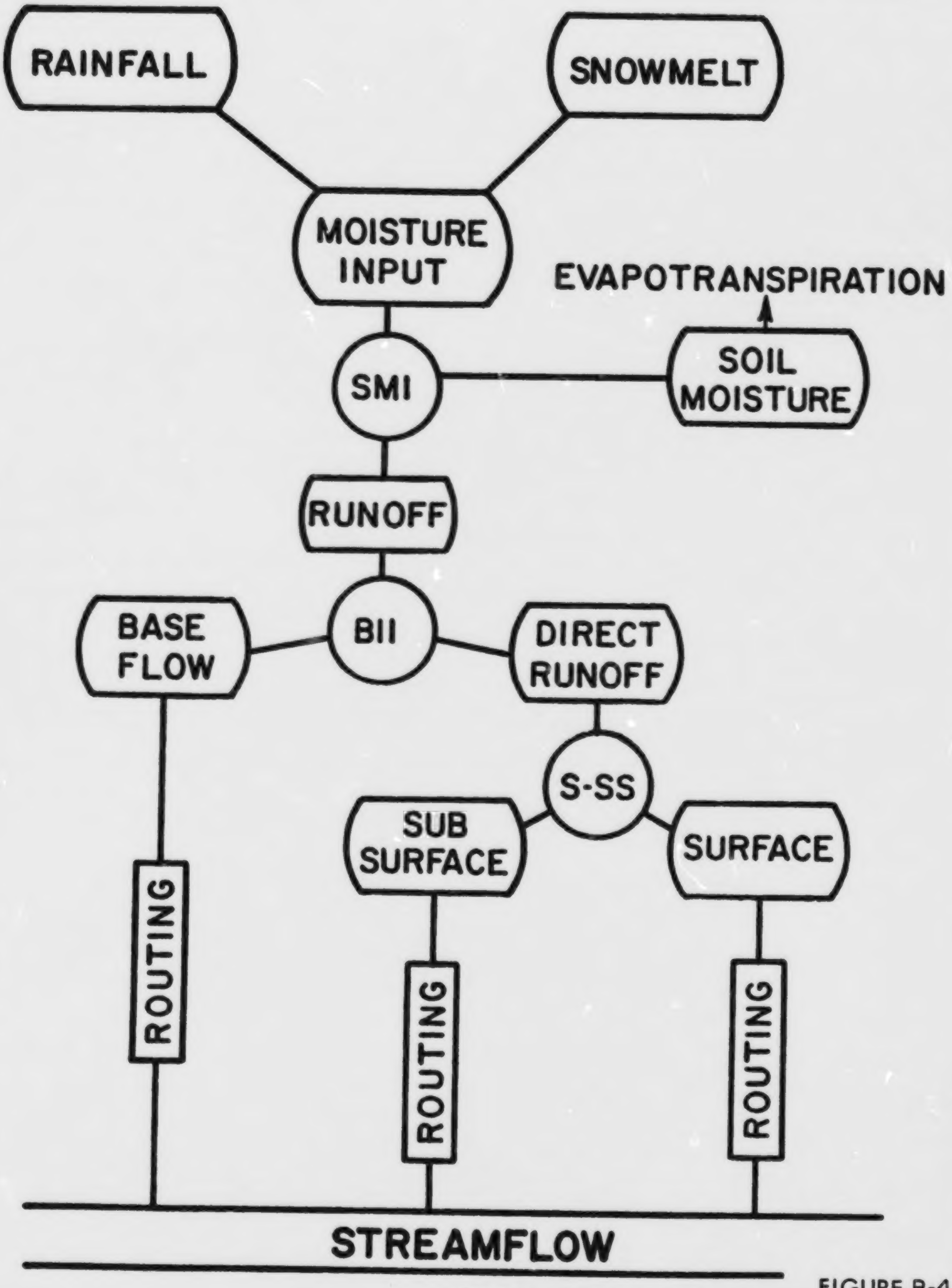
FIGURE B-3



RAINFALL
SNOWMELT
MOISTURE INPUT
EVAPOTRANSPIRATION
SMI
SOIL MOISTURE
RUNOFF
BASE FLOW
BII
DIRECT RUNOFF
S-SS
SUB SURFACE
SURFACE
ROUTING
ROUTING
ROUTING
STREAMFLOW

FIGURE B-4

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

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## BASIC PROCESSES OF THE SSARR MODEL

| | | | | | | | |
|---|---|---|---|---|---|---|---|
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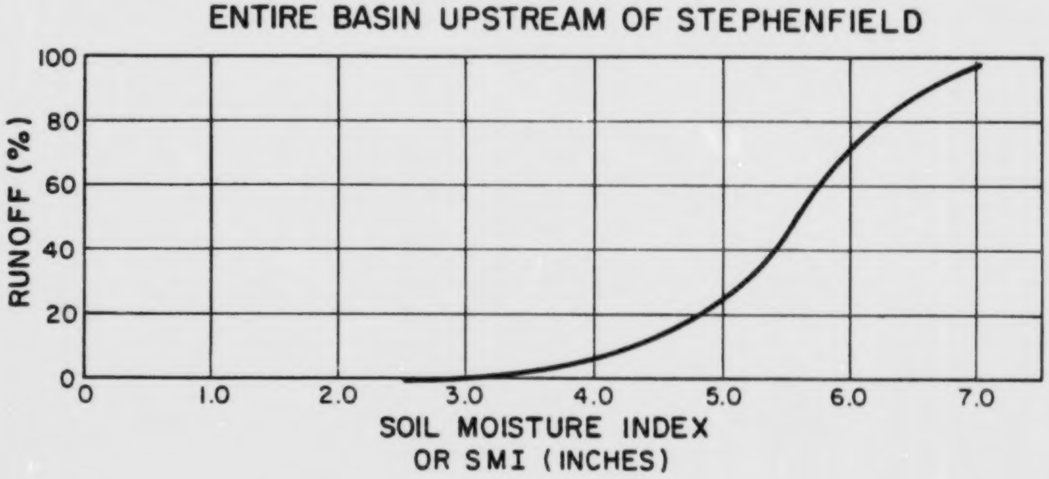

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CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## RUNOFF PERCENT CURVE
BASIN UPSTREAM OF STEPHENFIELD
SSARR MODEL

| | | | |
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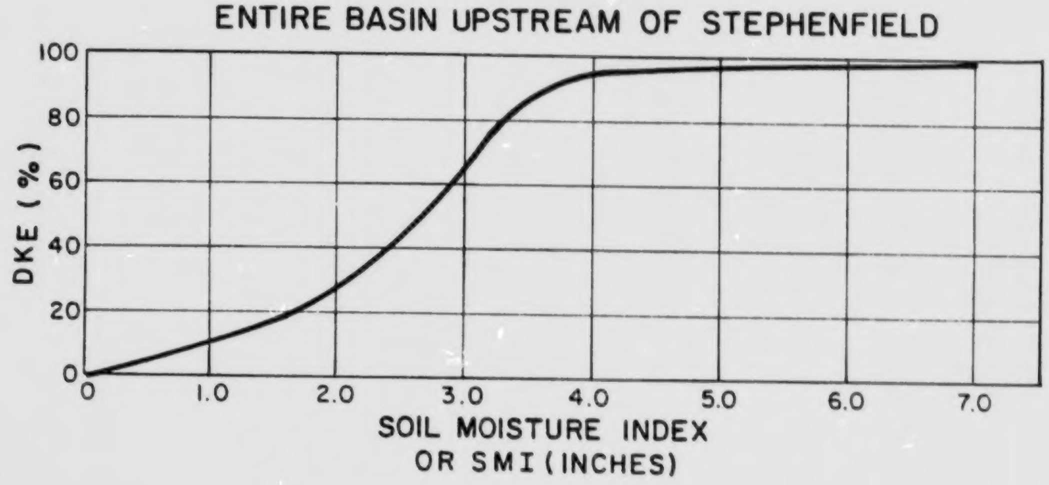
FIGURE B-5



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CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## SOIL DRYING CURVE
## BASIN UPSTREAM OF STEPHENFIELD
## SSARR MODEL

| | | | |
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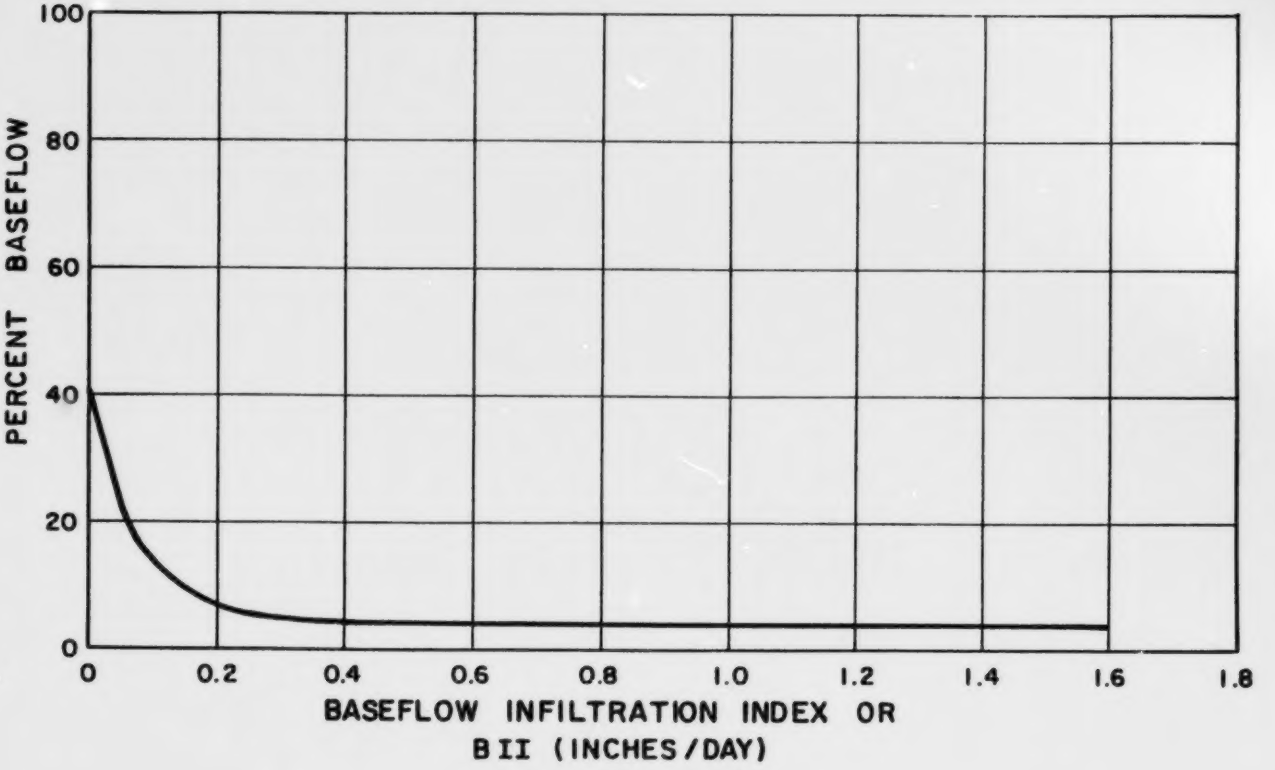
FIGURE B-6



**NOTE:**

DIRECT RUNOFF = (TOTAL RUNOFF) X (1-PERCENT BASEFLOW/100)

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| | | | |
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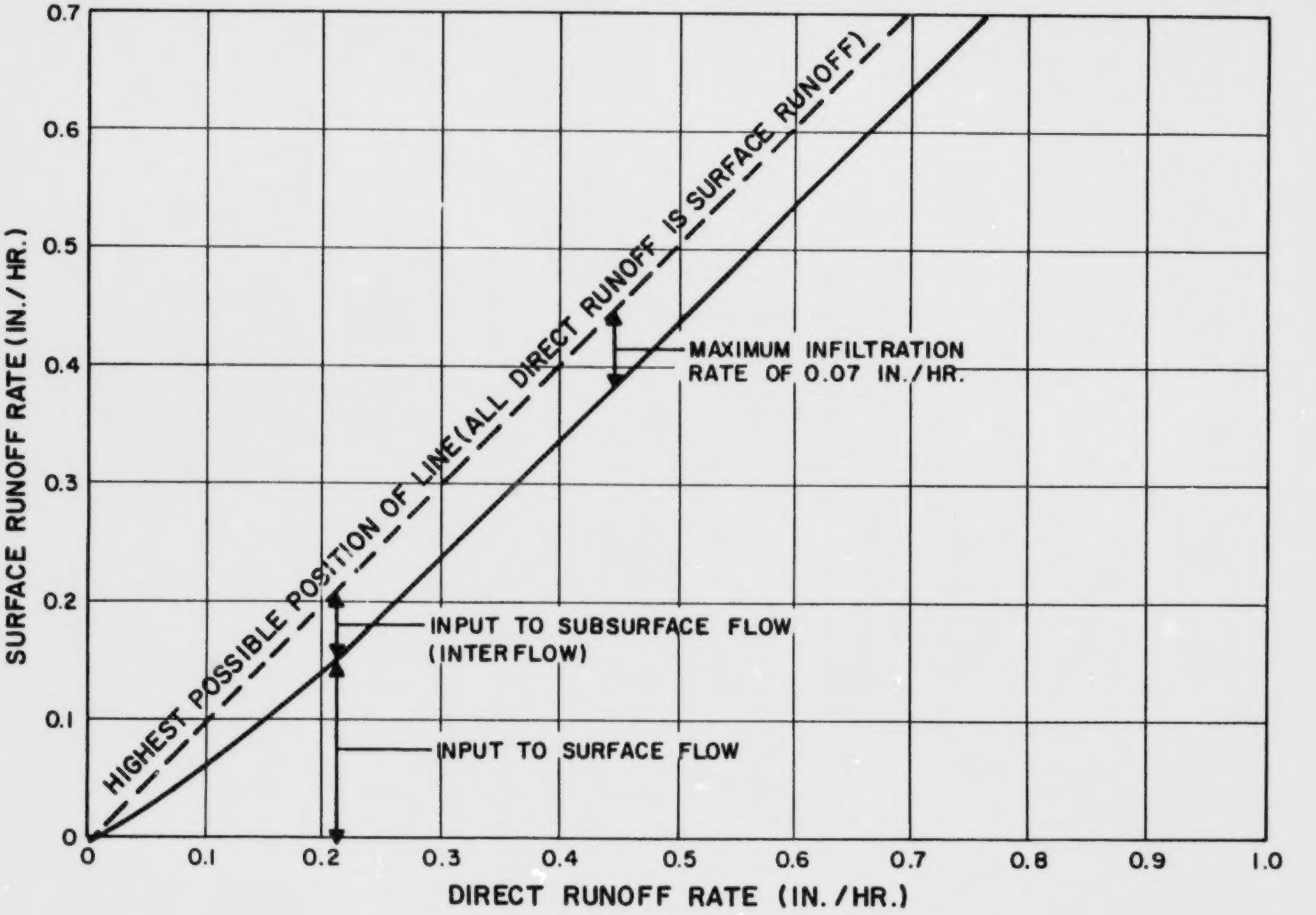
FLOOD FORECASTING AGREEMENT - PHASE ONE

## BASEFLOW PERCENT CURVE
## SSARR MODEL

| | | | |
|---|---|---|---|
| SCALE AS SHOWN | DATE | SHEET OF | FILE NO. |

FIGURE B-7



**NOTE:**

Sub-surface Runoff = Direct Runoff minus Surface Runoff.

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FLOOD FORECASTING AGREEMENT - PHASE ONE

## SURFACE / SUB-SURFACE RUNOFF CURVE
## S S A R R MODEL

| | | | | | | | |
|---|---|---|---|---|---|---|---|
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FIGURE B-8

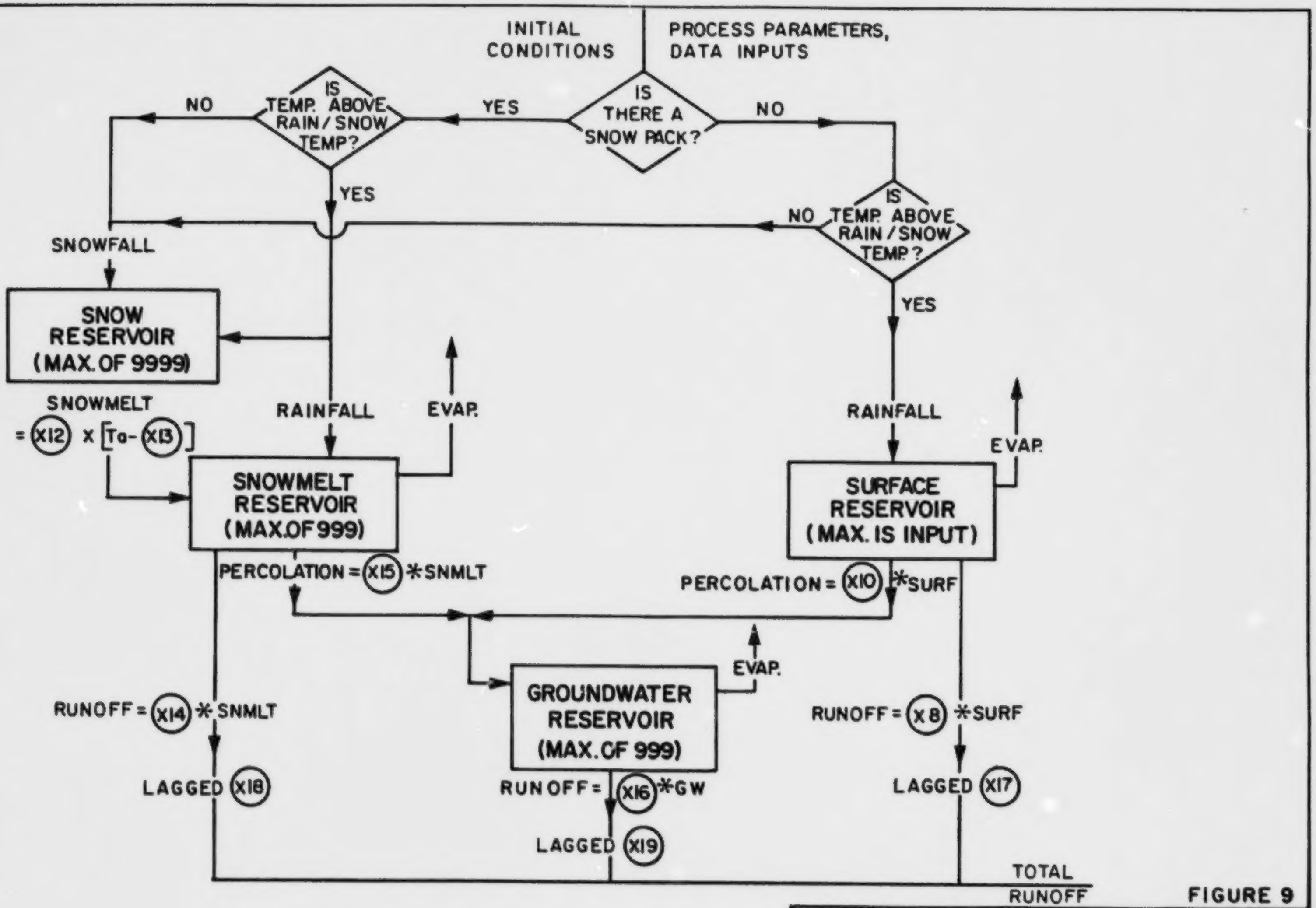
INITIAL CONDITIONS
PROCESS PARAMETERS, DATA INPUTS
IS THERE A SNOW PACK?
YES
NO
IS TEMP. ABOVE RAIN / SNOW TEMP.?
NO
YES
IS TEMP. ABOVE RAIN / SNOW TEMP.?
NO
YES
SNOWFALL
SNOW RESERVOIR (MAX. OF 9999)
SNOWMELT = X12 x [Ta- X13]
RAINFALL
EVAP.
SNOWMELT RESERVOIR (MAX. OF 999)
PERCOLATION = X15 *SNMLT
RAINFALL
EVAP.
SURFACE RESERVOIR (MAX. IS INPUT)
PERCOLATION = X10 *SURF
EVAP.
GROUNDWATER RESERVOIR (MAX. OF 999)
RUNOFF = X14 *SNMLT
LAGGED X18
RUN OFF = X16 *GW
LAGGED X19
RUNOFF = X8 *SURF
LAGGED X17
TOTAL RUNOFF

FIGURE 9

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

# BASIC CONFIGURATION
OF THE
# SLURP HYDROLOGIC MODEL

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FIGURE B-9

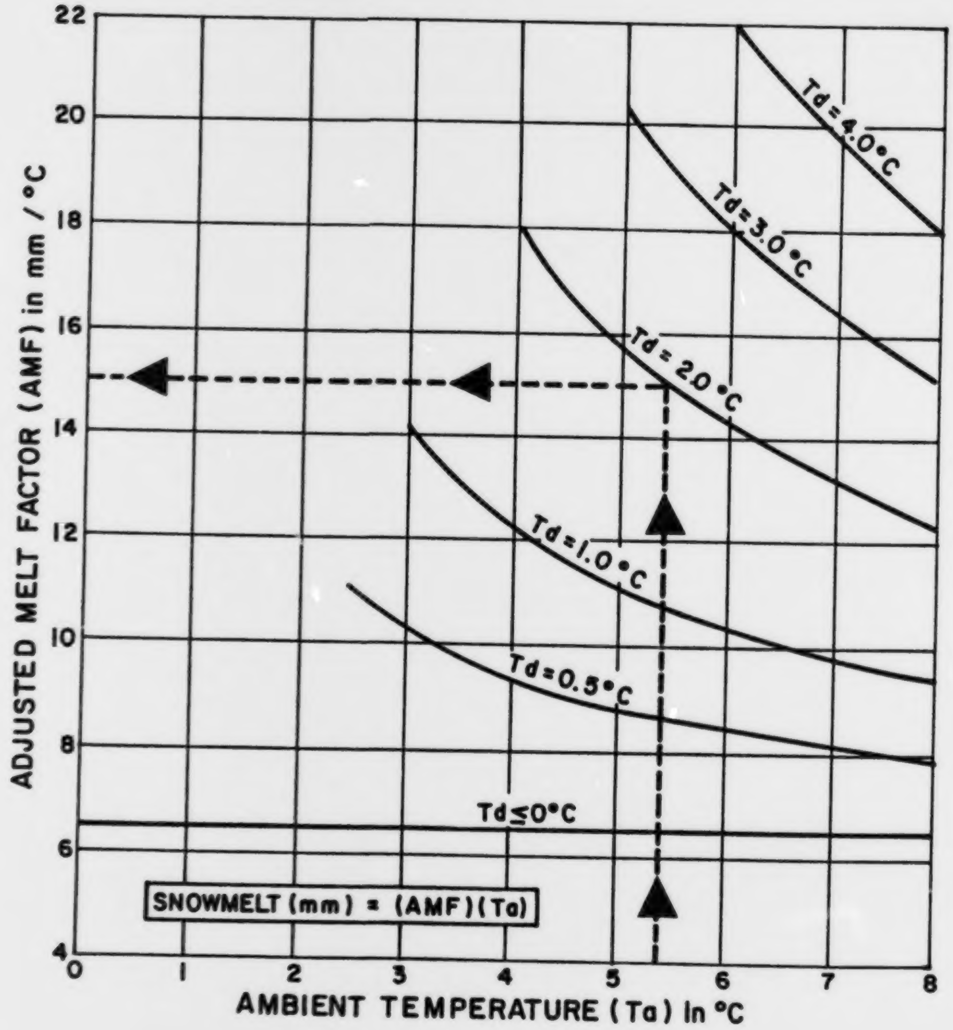

FIGURE B-10

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## ADJUSTMENT OF MELT FACTOR BASED ON DEW POINT TEMPERATURE-SLURP MODEL

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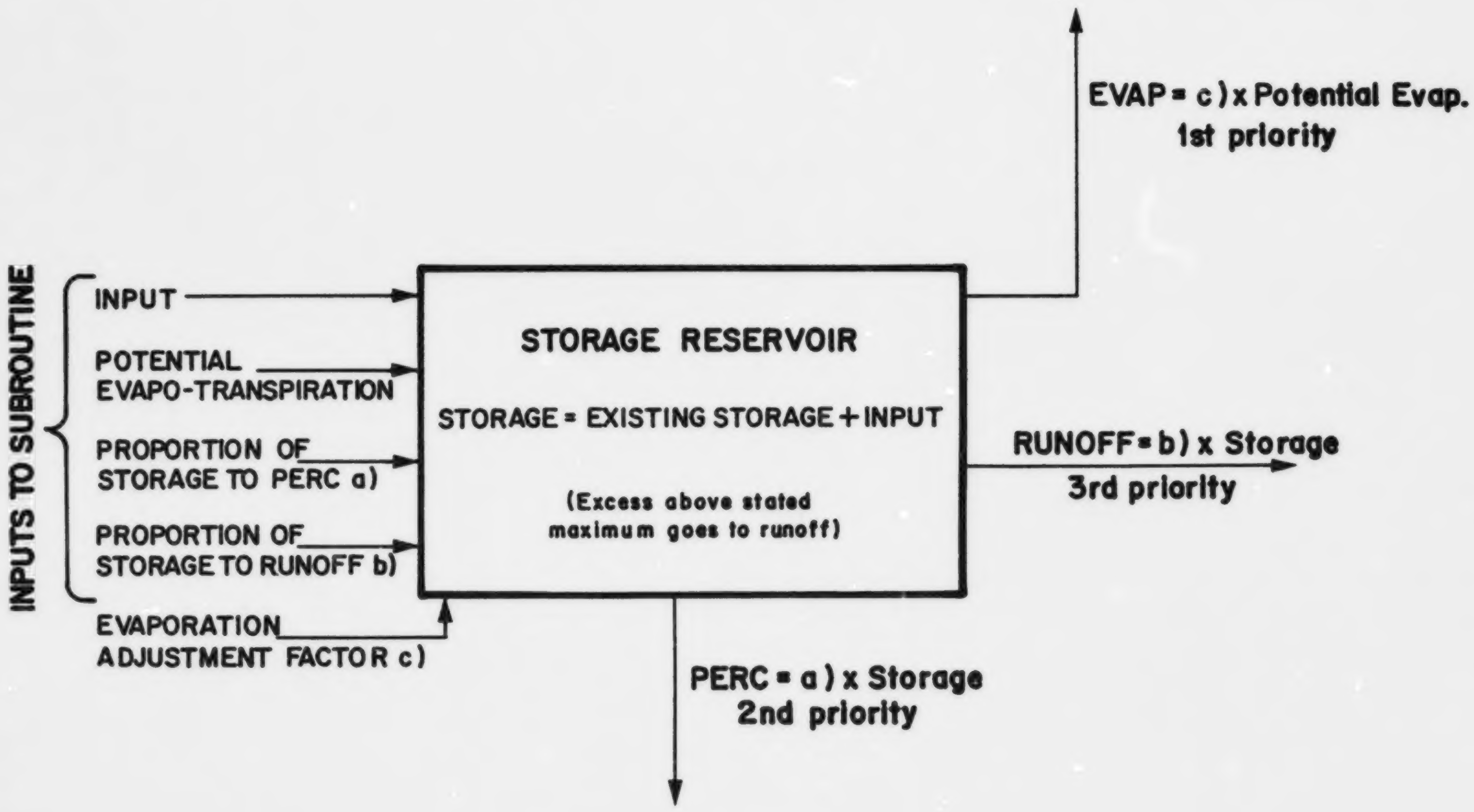
INPUTS TO SUBROUTINE
INPUT
POTENTIAL EVAPO-TRANSPIRATION
PROPORTION OF STORAGE TO PERC a)
PROPORTION OF STORAGE TO RUNOFF b)
EVAPORATION ADJUSTMENT FACTOR c)
STORAGE RESERVOIR
STORAGE = EXISTING STORAGE + INPUT
(Excess above stated maximum goes to runoff)
EVAP = c) x Potential Evap.
1st priority
RUNOFF = b) x Storage
3rd priority
PERC = a) x Storage
2nd priority

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CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

BASIC OPERATION OF A
STORAGE RESERVOIR
SLURP MODEL

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FIGURE B-11

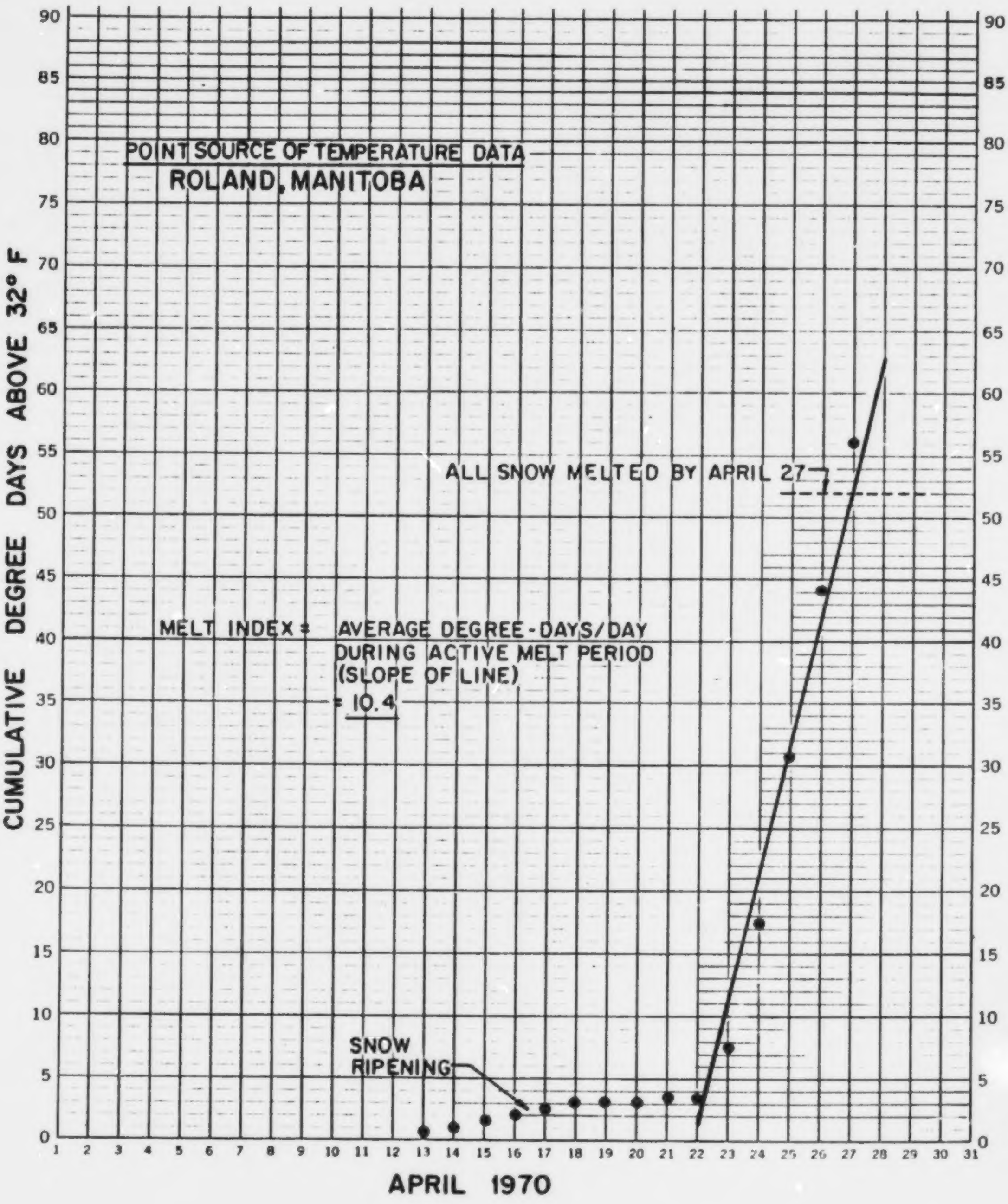

FIGURE B-12

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

# MELT INDEX CALCULATION

(BASIN UPSTREAM OF STEPHENFIELD)

## MANAPI MODEL

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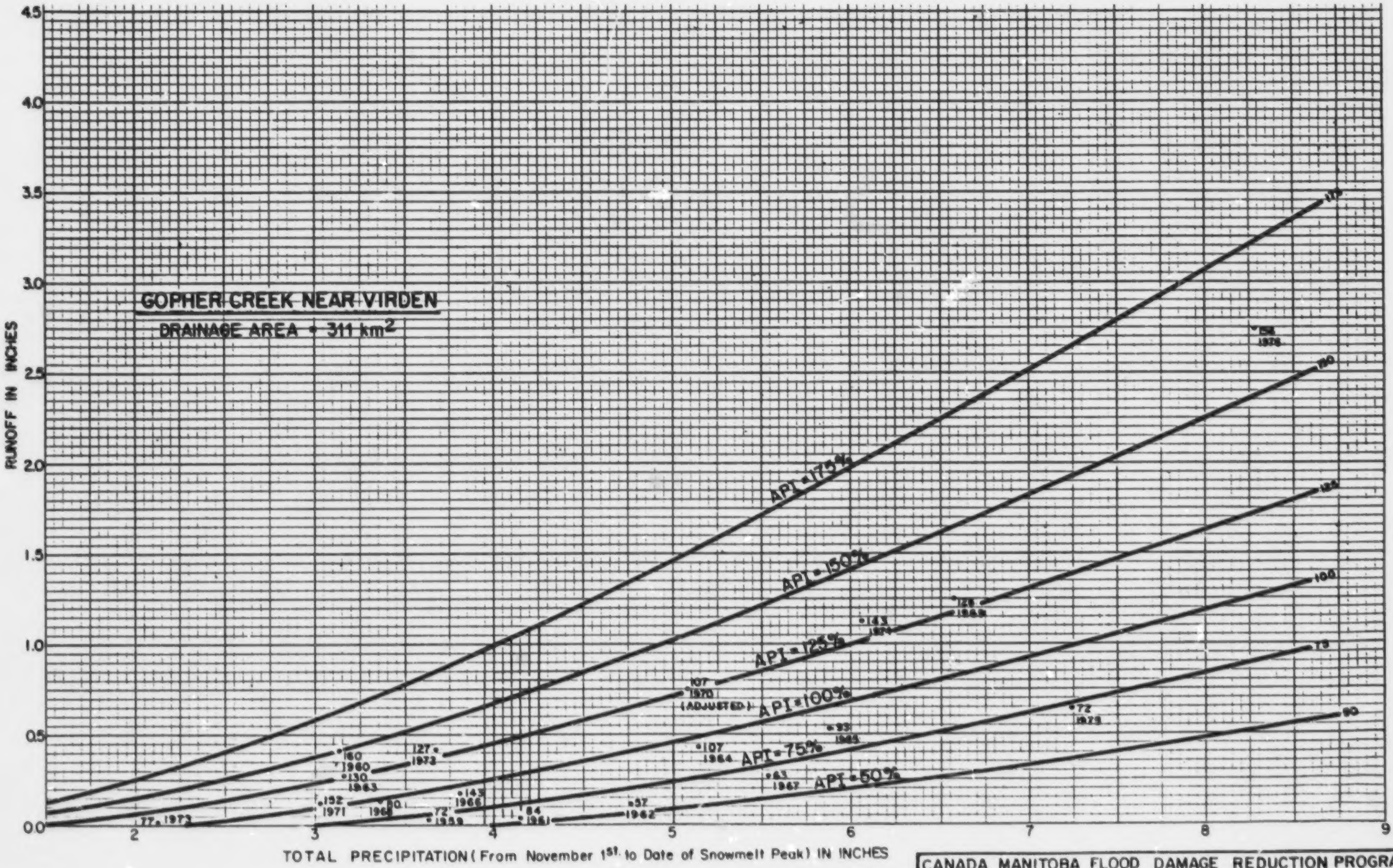

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CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## GRAPHICAL SNOWMELT RUNOFF FORECASTING - MANAPI MODEL

| | | | |
|---|---|---|---|
| PREPARED<br>A. A. W. | DRAWN<br>CHECKED | SUBMITTED | APPROVED |

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FIGURE B-13

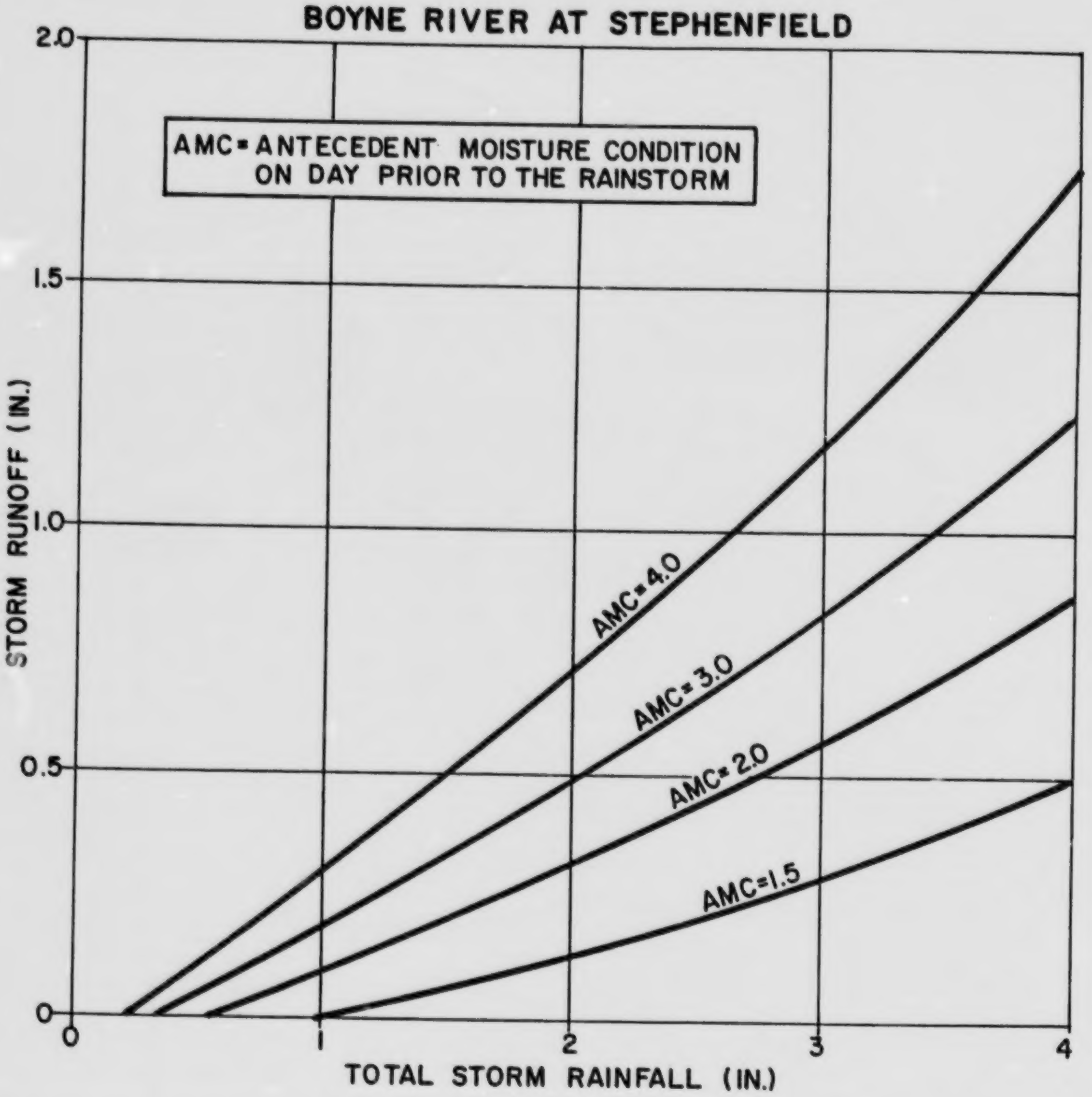
BOYNE RIVER AT STEPHENFIELD

FIGURE B-14

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## GRAPHICAL RAINFALL RUNOFF FORECASTING-MANAPI MODEL

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DEPARTMENT OF NATURAL RESOURCES
**WATER RESOURCES BRANCH**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>A.A.W. | DRAWN<br>CHECKED | SUBMITTED | APPROVED | SCALE<br>AS SHOWN | DATE | SHEET OF | FILE NO. |

## Appendix C

# THE BOYNE RIVER WATERSHED

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## C.1 LOCATION

The location of the Boyne River watershed is shown on Figure C-1. The most significant physiographic feature in the watershed is the Manitoba Escarpment which was formed following the retreat of the seas from central North America about 50 million years ago. The escarpment formed the shoreline of glacial Lake Agassiz during the end of the last ice age. As with many streams in this region the headwaters of the Boyne River are located on the plateau above the Manitoba escarpment. Below the escarpment the river flows eastward across the lacustrine plain and through the Town of Carman, located 82 kilometres southwest of the City of Winnipeg. The Boyne River continues eastward and ultimately enters the Red River at Morris by way of the Norquay Channel and the Morris River.

Carman with a population of 2,800 is the largest urban centre in the watershed. The second largest is Treherne, having a population of approximately 800.

## C.2 PHYSICAL FEATURES OF THE WATERSHED

### C.2.1 Dimensions and Drainage Area

The maximum dimensions of the watershed are 65 kilometres east to west and 45 kilometres north to south. The gross drainage area upstream of the hydrometric gauge near Carman is 1,151 $km^2$ , of which 881 $km^2$ lie upstream of the hydrometric gauge near Stephenfield. The main channel is 151 kilometres in length while the largest tributary, Roseisle Creek, is 41 kilometres long.

### C.2.2 Relief

Three distinct topographic regions can be delineated within the Boyne River watershed on the basis of relief, as illustrated on Figure C-2.

The largest of these is a relatively flat plain located on the lowland portion of the basin below the 335 metre contour. This plain comprises 57 percent of the basin. The low relief in this portion of the watershed is primarily due to a layer of lake clays which underlay the surface deposits of deltaic sands and other sediments. The surface slope of the land is as low as 0.34 percent.

Above the 450 metre contour interval is a region of upland plateau which forms 22 percent of the basin's area. Sometimes referred to as the second prairie level, this region is characterized by an undulating, pothole dotted landscape. This is typical of an area where glaciation has been the primary geomorphic agent.

The transition between the plateau and the lowland plain is marked by a steeply sloping escarpment. Although this region is only 21 percent of the total basin area, it is a major contributor to streamflow due to land surface slopes averaging as high as 2.1 percent in many areas.

A streambed profile of the Boyne River is shown on Figure C-3. The Boyne River channel itself does not exhibit the steep escarpmental slopes, since it generally meanders along the base of the escarpment, where it is joined by tributaries flowing down the escarpment from the plateau.

### C.2.3 Surficial Deposits

The rate at which water can pass from the surface soil into and through the subsurface materials is largely a function of the texture and depth of sediments overlying the bedrock. The sediments are referred to as surficial deposits or overburden since they are found on the surface of the bedrock. They are distinguished from surface soils in that they do not contain sufficient organic material to be classified as a soil. The areal distribution of these deposits is shown on Figure C-4.

The basin's lowlands are covered largely by deltaic sands from two phases of a delta formed when the Assiniboine River discharged into Lake Agassiz. Directly north of the escarpment are medium to fine grained sands about eight metres thick. East of the escarpment are sediments deposited at a later time, when the lake level was lower. These vary from six metres of medium grained sand in the west to less than a third of a metre of very fine sand on the delta's eastern edge, near Carman.

The delta deposits are underlain by Lake Agassiz clays. The clay layer increases in depth with distance away from the escarpment and is greater than 45 metres thick near Carman. The heavy texture of the clay resists the vertical movement of water that has passed through the highly permeable deltaic sands, thereby increasing runoff potential.

Lacustrine silty clays are found east of Rathwell where the Boyne River has been forced to turn southward along the edge of the lower delta. The ancient lake that resulted caused the deposition of 2.5 metres of silty clay over a metre of deltaic sands, underlain by lake clays. It is an area of high runoff potential due to the low permeability of the silty clays.

The escarpment and plateau are covered by heavy textured deposits of either a glacial or glacio-fluvial origin. These sediments have a low permeability due to a predominance of clay in their texture.

The land surface of the area shown as till is characteristic of the glaciated portions of the prairies. The land surface is rolling and dotted with many small depressions and ponds, left as fragmented glacial ice melted in place. These depressions actively

**contribute to overland flow only during periods of high moisture. The typical profile consists of 4.5 metres of clay till underlain by two metres of unconsolidated weathered shale over shale bedrock.**

**In areas where meltwater has caused the sorting and redeposition of till the land surface is more uniform and has fewer closed depressions. Typically these deposits consist of four metres of clay over three metres of either sand, weathered shale, or till, underlain by shale bedrock.**

**A wide variety of beach and alluvial deposits are found along the base of the escarpment. The conveyance of water is affected by both the texture of these deposits, which tends to be coarse, and by the fact that they appear as ridges running at right angles to the flow path. As such, these deposits increase the overall complexity of the basin for modelling purposes.**

### C.2.4 Soils

**The pattern of surface soils as shown in Figure C-5 closely resembles that of the surficial deposits.**

**The soils of the escarpment and plateau are heavy textured due to the high percentage of clay in the parent material (Ellis and Shafer, 1943). Pembina clay loam has been formed on till that has remained relatively unchanged since glaciation. Where meltwater has caused sorting of the till, a soil of higher clay content has formed.**

**Infiltration rates for both these soils are low and the potential exists for this area to produce large amounts of runoff.**

**The soils formed on deltaic deposits include the Almassippi complex which is composed of sand to very fine sandy loam, the Holland very fine sandy loam and the Stockton loamy sands. These tend to have a light texture and relatively high infiltration rates. Coarser soils are found at the upstream side of each delta complex. The soils become both finer and thinner as one goes farther out in the deltas.**

**Rathwell silty clays have developed near Rathwell on the lacustrine sediments deposited where the river has been forced to turn south along the edge of the lower delta (Ehrlich et al, 1957). Infiltration into this soil is inhibited by its heavy texture.**

**In the centre of the Rathwell silty clays is an area of marsh soils which are indicative of poor drainage and frequent flooding along this reach of the river.**

**Soils that have developed on beach deposits at the base of the escarpment contain concentrations of sand and gravel. Infiltration rates are not easily identified due to significant local variations in the texture of the upper soil horizon.**

**A fairly extensive area of soil has formed on the shale-clay fragments deposited at the base of the escarpment near Roseisle Creek. This soil is called the Morden complex. Movement of water into this soil is controlled primarily by the subsurface conditions. It is well drained if the coarser material has substantial depth and poorly drained if a clay substrate is present. Both phases are found in the Boyne River watershed.**

### C.2.5 Vegetation

Over half of the land surface within the Boyne River watershed is under active cultivation. Most of the crops consist of cereal grains in rotation with fallow. Woodlands and pasture are usually found where either the soil type or the topography are unsuitable for grain or other cash crops.

Woodlands are predominant along the eroded valleys of creeks draining the escarpment and along the stony beach ridges at the base of the escarpment. Agricultural use of the land is restricted on much of the land underlain by Almassippi Complex soils, denoted by Als and Alls on Figure C-5, due to the low water holding capacity of these soils and due to their susceptibility to wind erosion. The western portion of this soil complex is used almost exclusively as pasture or has been left as woodland. Pasture lands on these soils are predominantly planted forage crops, although pastures of native grasses and scrub trees have been left in areas of high erosion potential. As the soils become finer in texture towards the east the amount of land under cultivation increases.

### C.2.6 Watershed Segments

The Boyne River watershed was divided into six sub-areas or segments, each of which had unique physiographic characteristics. This was done in order to test whether flows in the Boyne River could be better simulated by modelling each segment separately and then routing the simulated segment flows down the main river channel.

The six watershed segments were delineated as shown on Figure C-6. Following is a description of each of the segments.

Segment 1 has a gross drainage area of 194 $km^2$ . Due to the pothole topography the effective drainage area is less than 75 percent of the gross area in all but the wettest conditions. The segment is located on the edge of the escarpment and has a relatively steep slope of .0132. The lower half of this segment is comprised of the Upper Boyne Marsh. The basin is moderately forested. The soil material consists of the type of glacial till which has a low infiltration capacity.

Segment 2 has a gross drainage area of 60 $km^2$ , most of which contributes to runoff under relatively wet conditions. The segment is located at the base of the escarpment and has a milder slope of .0073. There is relatively little tree cover and extensive farming throughout the segment. The soil consists mainly of sand deposits which have a high infiltration capacity.

Segment 3 has a gross drainage area of 171 $km^2$ , most of which contributes to runoff under relatively wet conditions. The segment is located at the base of the escarpment. The upstream half of the basin is very flat and encounters large amounts of standing water during floods. The downstream half of the segment has better relief and less standing water. The basin has no tree cover except along the stream channel. The soils are lacustrine deposits which have low infiltration rates.

Segment 4 has a gross drainage area of 107 $km^2$ . Virtually the entire area contributes to runoff in all but the driest conditions. The segment is located at the edge of the escarpment and has a steep basin slope of .026. The lower elevation of the segment is farmed. The upper elevation is partially tree covered and not extensively farmed. The soil is a glacial till which has a low infiltration rate.

Segment 5 has a gross drainage area of 155 $km^2$ . It is quite flat, but due to an extensive network of man-made drains most of it contributes to runoff under relatively wet conditions. The segment has an average slope of .0055. The segment includes the Stephenfield Reservoir. The land is mostly farmed, but there is also some tree cover and pasture land. The segment consists of stable sand dunes covered by a thin layer of soil and thus has a high infiltration capacity.

Segment 6 consists of the largest tributary to the Boyne River, namely Roseisle Creek. The basin has a gross drainage area of 194 $km^2$ , of which 168 $km^2$ usually contribute to runoff under relatively wet conditions. The downstream portion of the segment is located on the edge of the escarpment and has a relatively steep slope, while the upstream portion is relatively flat. The soil is predominantly composed of glacial till which has a low infiltration capacity.

## C.3 HYDROMETRIC NETWORK

The hydrometric data collection network within the Boyne River watershed consists of five streamflow gauges and one lake level recorder (Surface Water Data Reference Index, 1983). The location of these gauges is shown on Figure C-7. Streamflow gauges on the main stem are located near Treherne, Roseisle, Stephenfield and Carman. Roseisle Creek flows are gauged near the Town of Roseisle. A water level recorder is located on Stephenfield Reservoir.

Records have been kept for the Boyne River near Carman sporadically from 1919 to 1930 and regularly from 1956 to the present (1984). A recording gauge was installed in 1976.

Manual gauges were installed on the Boyne River near Treherne and near Rathwell in 1967 and were converted to automatic recording gauges in 1969.

The streamflow gauge near Stephenfield has been in operation since 1952 and has been equipped with a recorder since the summer of 1969.

Roseisle Creek has been gauged since 1965. An automatic recorder has been in operation on this Creek since 1974.

The level of Stephenfield Reservoir was read manually from 1964 until 1978 when a recording gauge was installed.

## C.4 CLIMATOLOGICAL NETWORK

Figure C-8 shows the locations of climate stations in and around the Boyne River watershed. Although not all climate stations collect the same data, maximum and minimum daily temperatures and daily precipitation are generally recorded. Only the Deerwood station measures rainfall intensity. The number of climate stations has varied from year to year; not all of the station on Figure C-8 have reported in any given year (Climatological Station Data Catalogue, 1981).

Further information on the climatological network pertinent to the Boyne River watershed is contained in Appendix D, "Preparation of Data Inputs for Hydrologic Models".

## C.5 HISTORY OF FLOODS

The Boyne River has a recurring flooding problem. The Town of Carman has been inundated with flood waters seven times in the last 100 years. Most of these floods resulted from abnormally high runoff from spring snowmelt, sometimes augmented by spring rain. Details are shown in Table C-1.

The damage caused by major floods on the Boyne River is widespread. Near the Town of Rathwell flood waters can form a lake several miles long and up to a mile wide. Most of this land is cultivated and farmers in the area face the hardship of delayed seeding and erosion of cropland.

A more severe problem exists at Carman. For aesthetic reasons and due to a lack of floods between 1923 and 1969 a large part of the Town has been developed on the flood plain of the river. As a result, extensive damage is suffered during major floods. Damage to structures alone during the 1979 flood have been estimated at $643,000 in 1979 dollars. The 1974 spring flood was very similar in magnitude to that of 1979, while the flood of 1970 was of lesser magnitude.

Flooding occurs in Carman when the channel capacity of 68 $m^3/s$ is exceeded. The return period of a flow of this magnitude to Carman is six years based on a flood frequency analysis of recorded peak flows and estimated historic peak flows. A flood peak of the magnitude of the 1979 event, i.e. 116 $m^3/s$ , has been estimated to have a return period of 18 years.

## C.6 LITERATURE CITED

1. Ellis, J.H. and Shafer, Wm.H., 1943. Reconnaissance Soil Survey of South-Central Manitoba. Manitoba Soil Survey.

2. Ehrhich, W.A., Poyser, E.A. and Pratt, L.E., 1975. Reconnaissance Soil Survey of Carberry Map Sheet Area. Manitoba Soil Survey.

3. Surface Water Data Reference Index-Canada, 1983. Inland Waters Directorate, Water Resources Branch, Water Survey of Canada, Ottawa.

4. Climatological Station Data Catalogue, 1981. Atmospheric Environment Service, Environment Canada, Downsview, Ontario.

## Table C-1
## History of Floods on the Boyne River
## (Based on Hydrometric Reports for the Boyne River near Carman)

| Year | Peak Discharge ($m^3/s$) (Mean Daily) | Date | Primary Cause of Flood |
|---|---|---|---|
| *1893 | 187 | Early July | Heavy rain |
| **1896 | ? | Early May | Snowmelt, heavy rain |
| 1923 | 70.5 | April 22 | Rapid snowmelt |
| 1969 | 69.7 | April 15 | Rapid snowmelt |
| 1970 | 105 | April 30 | Rapid snowmelt |
| 1974 | 132 | April 22 | Rapid snowmelt, rain |
| 1979 | 119 | April 23 | Rapid snowmelt |

* Discharge estimated from stages recorded by Sullivan Survey. Time of occurrence estimated from climatological data.

** Estimate of discharge is unavailable, but a historical reference suggests that this flood was greater than that of 1893. Time of occurrence estimated from climatological data.

Note: Bankfull flow on the Boyne River at Carman is approximately 68 cubic metres per second.

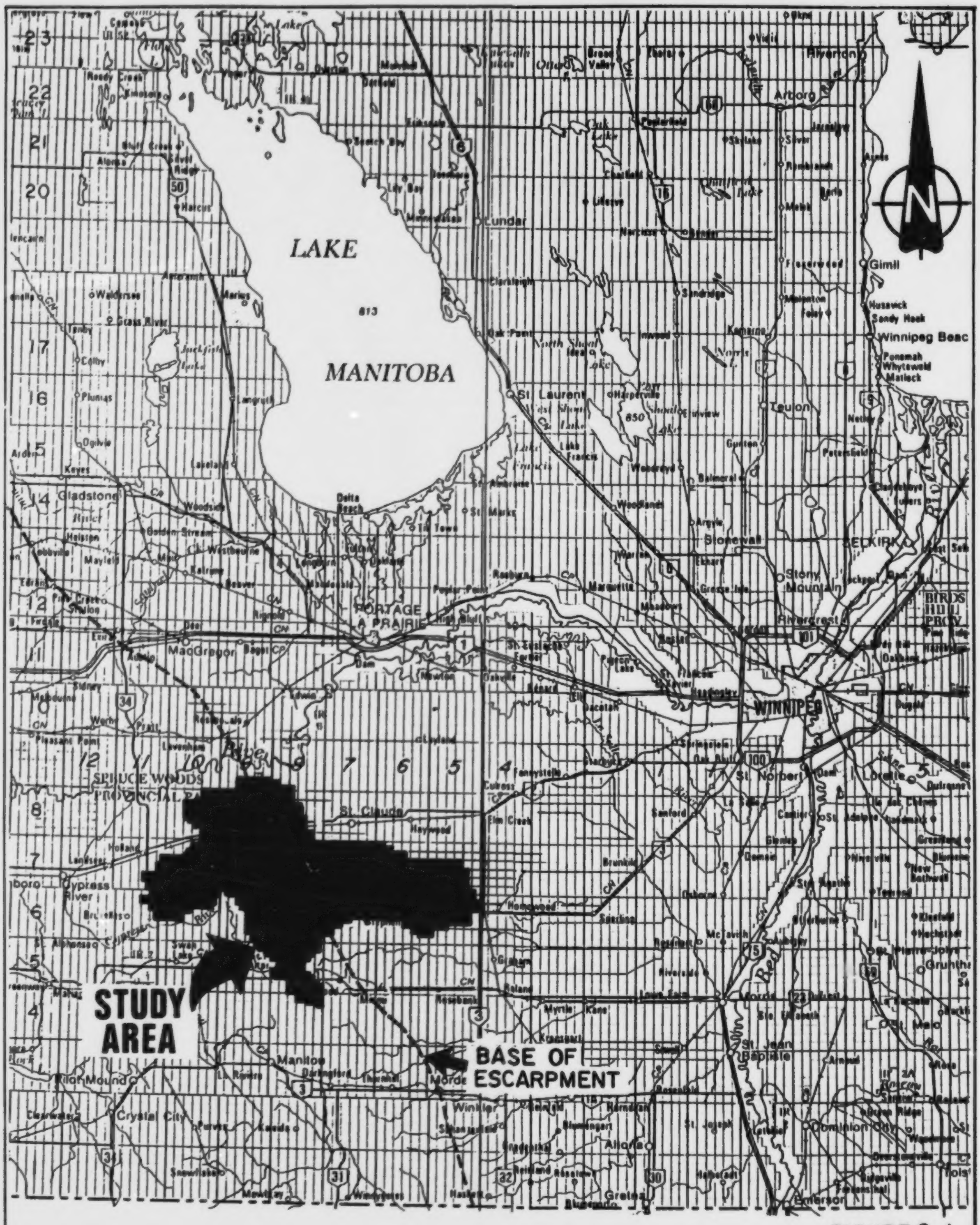

LAKE MANITOBA
STUDY AREA
BASE OF ESCARPMENT
N
WINNIPEG
PORTAGE LA PRAIRIE
SELKIRK
Gimli
Riverton
Arborg
Teulon
Stonewall
Stony Mountain
St. Laurent
Gladstone
MacGregor
Carman
St. Claude
Morris
St. Jean Baptiste
Emerson
Altona
Winkler
Morden
Manitou
Pilot Mound
Crystal City
Cypress River
Holland
SPRUCE WOODS PROVINCIAL PARK
Dominion City
St. Norbert
St. Adolphe
Ste. Agathe
Lundar
Oak Point
Delta Beach
Winnipeg Beach

FIGURE C-1

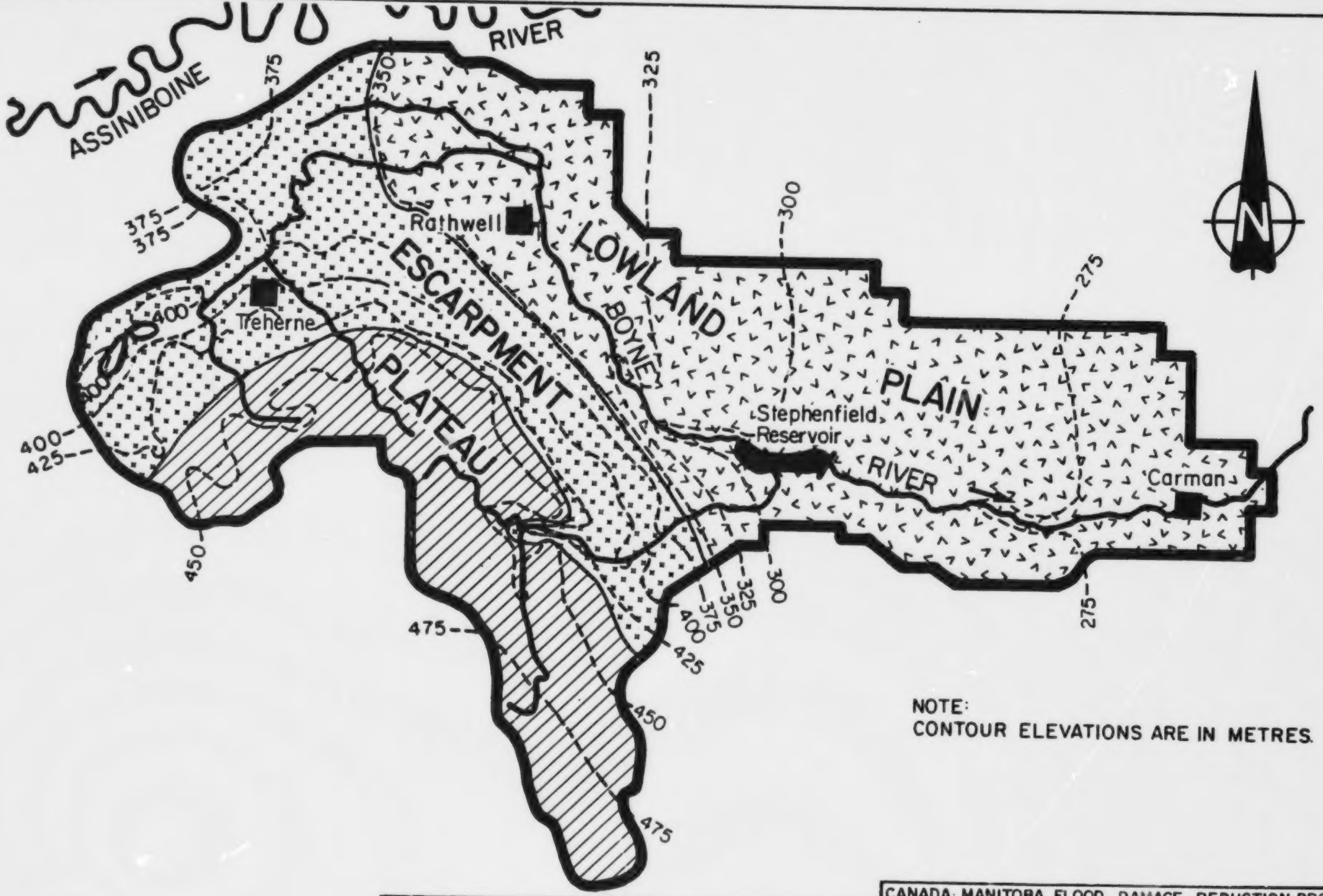
CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

## BOYNE RIVER WATERSHED
## LOCATION PLAN

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| A.A.W. | CHECKED | | | AS SHOWN | | OF | |


ASSINIBOINE
RIVER
375
350
325
300
275
375
375
400
400
400
425
450
475
425
400
375
350
325
300
275
450
475
Rathwell
Treherne
LOWLAND
ESCARPMENT
PLATEAU
BOYNE
PLAIN
Stephenfield
Reservoir
RIVER
Carman
N

NOTE:
CONTOUR ELEVATIONS ARE IN METRES.

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

| | | | |
|---|---|---|---|
| PREPARED<br>E.L.B. | DRAWN<br>CHECKED A.W. | SUBMITTED | APPROVED |

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

# RELIEF MAP
# BOYNE RIVER WATERSHED

| | | | |
|---|---|---|---|
| SCALE<br>AS SHOWN | DATE | SHEET OF | FILE NO. |

FIGURE C-2

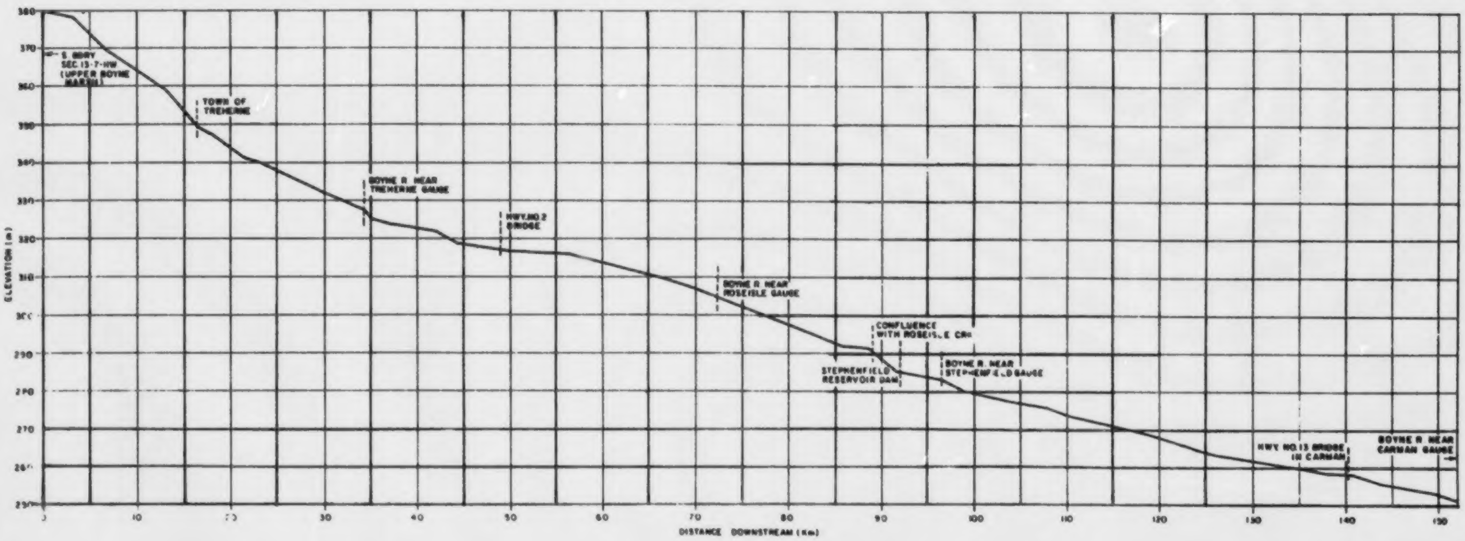


FIGURE C-3

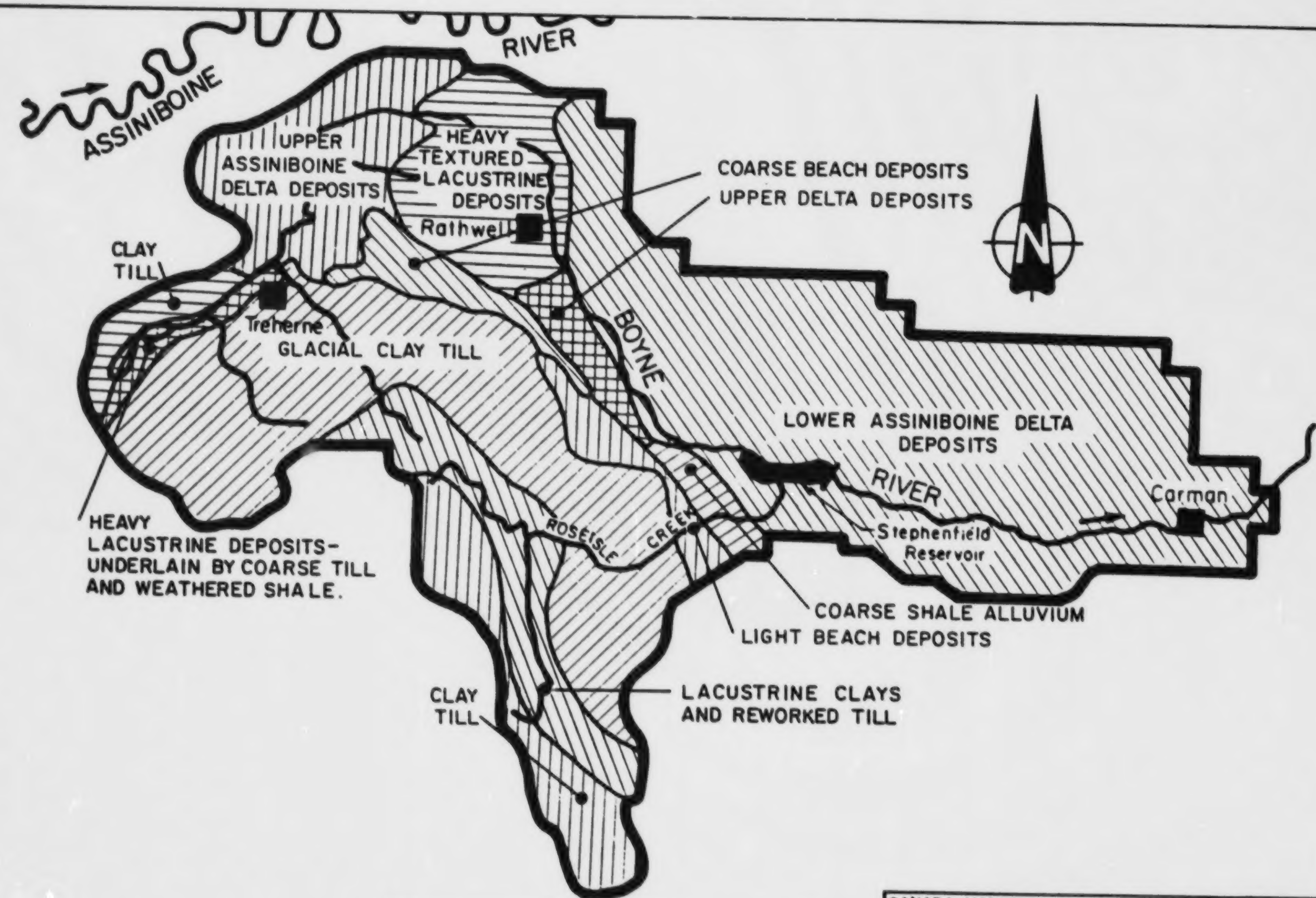
| | |
|---|---|
| DRAWN<br>J. Memon | PROVINCE OF MANITOBA<br>DEPARTMENT OF NATURAL RESOURCES<br>WATER RESOURCES BRANCH |
| CHECKED<br>A.A.N. | |
| PREPARED<br>E.L.G. | SUBMITTED HF APPROVED |

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE ONE

**STREAMBED PROFILE**
OF THE
**BOYNE RIVER**

| SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|
| AS SHOWN | 84.09.01 | 1 OF 1 | |


ASSINIBOINE
RIVER
UPPER ASSINIBOINE DELTA DEPOSITS
HEAVY TEXTURED LACUSTRINE DEPOSITS
COARSE BEACH DEPOSITS
UPPER DELTA DEPOSITS
N
CLAY TILL
Rathwell
Treherne
GLACIAL CLAY TILL
BOYNE
LOWER ASSINIBOINE DELTA DEPOSITS
RIVER
Carman
Stephenfield Reservoir
ROSEISLE
CREEK
HEAVY LACUSTRINE DEPOSITS- UNDERLAIN BY COARSE TILL AND WEATHERED SHALE.
COARSE SHALE ALLUVIUM
LIGHT BEACH DEPOSITS
CLAY TILL
LACUSTRINE CLAYS AND REWORKED TILL

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WATER RESOURCES BRANCH

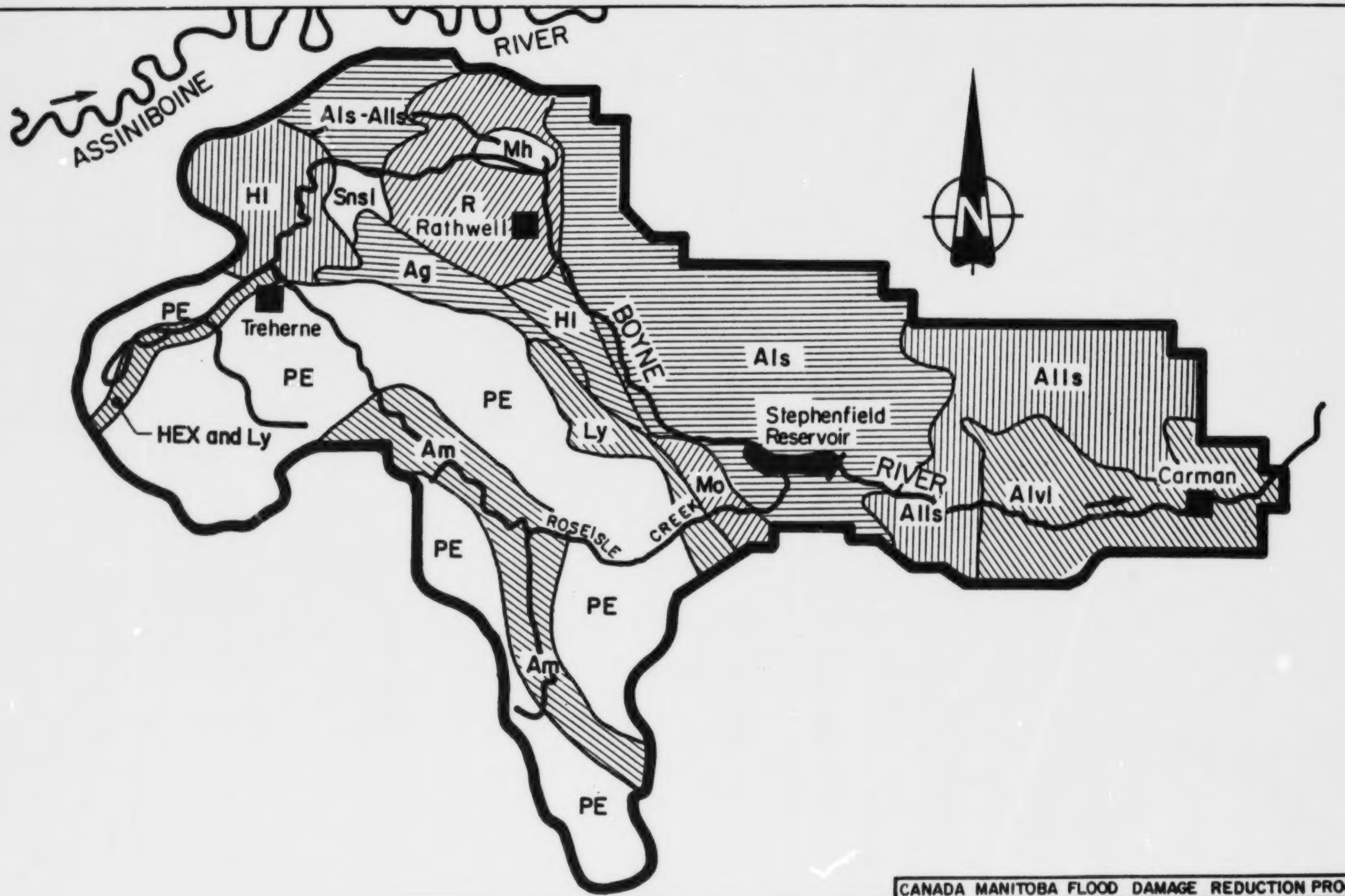
PREPARED E.L.B.
DRAWN
CHECKED A.W.
SUBMITTED
APPROVED

CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## BOYNE RIVER WATERSHED
## SURFICIAL DEPOSITS

SCALE AS SHOWN
DATE
SHEET OF
FILE NO.

FIGURE C-4


ASSINIBOINE
RIVER
Als-Alls
Mh
Hl
Snsl
R
Rathwell
Ag
PE
Treherne
PE
HEX and Ly
Hl
BOYNE
Als
Alls
PE
Ly
Stephenfield
Reservoir
RIVER
Am
Mo
Alls
Alvl
Carman
ROSEISLE
CREEK
PE
PE
Am
PE
N

CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## BOYNE RIVER WATERSHED
## SOIL TYPES

PROVINCE OF MANITOBA
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WATER RESOURCES BRANCH

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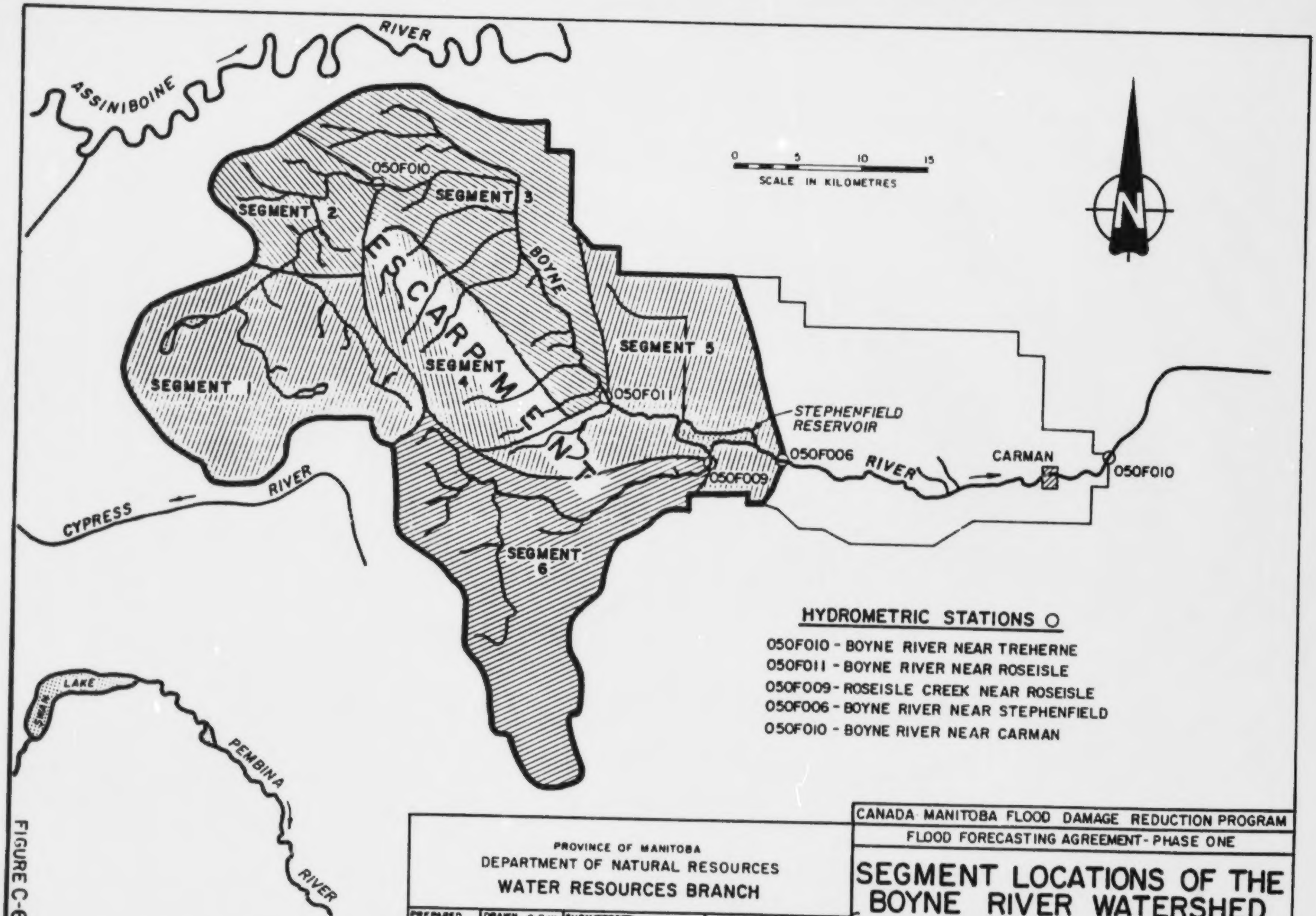
|---|---|---|---|---|---|---|---|
| E.L.B. | CHECKED A.W. | | | AS SHOWN | | OF | |

FIGURE C-5


RIVER
ASSINIBOINE
050F010
SEGMENT 2
SEGMENT 3
ESCARPMENT
BOYNE
SEGMENT 5
SEGMENT 4
SEGMENT 1
050F011
STEPHENFIELD RESERVOIR
050F006
RIVER
CARMAN
050F010
050F009
CYPRESS
RIVER
SEGMENT 6
0
5
10
15
SCALE IN KILOMETRES
N
LAKE
SWAN
PEMBINA
RIVER
HYDROMETRIC STATIONS ○
050F010 - BOYNE RIVER NEAR TREHERNE
050F011 - BOYNE RIVER NEAR ROSEISLE
050F009 - ROSEISLE CREEK NEAR ROSEISLE
050F006 - BOYNE RIVER NEAR STEPHENFIELD
050F010 - BOYNE RIVER NEAR CARMAN

PROVINCE OF MANITOBA
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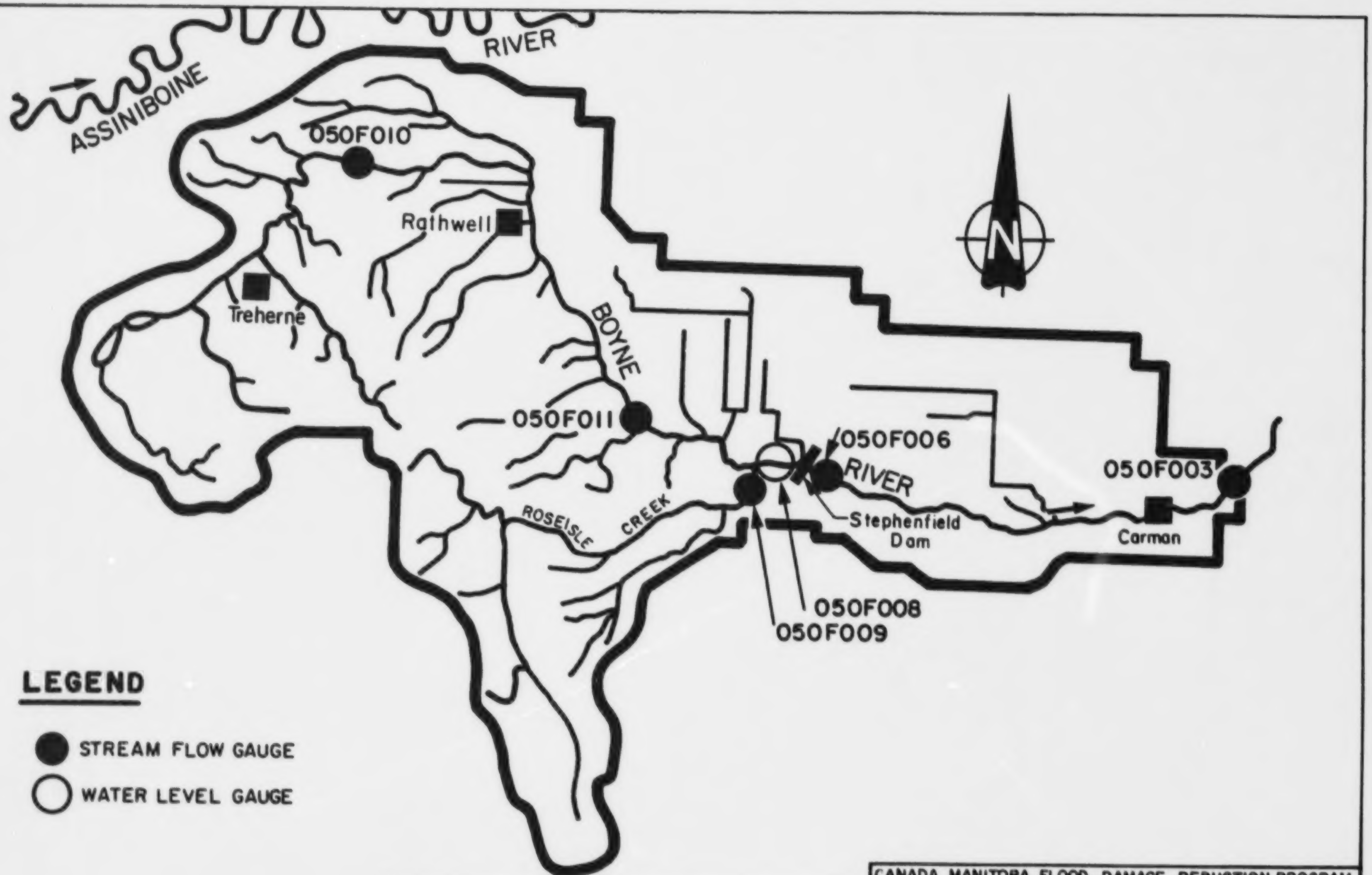
CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## SEGMENT LOCATIONS OF THE BOYNE RIVER WATERSHED

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED G.E.M. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE | SHEET OF | FILE NO. |

FIGURE C-6


ASSINIBOINE
RIVER
050F010
Rathwell
Treherne
BOYNE
050F011
ROSEISLE
CREEK
050F006
RIVER
Stephenfield Dam
050F008
050F009
050F003
Carman
N
LEGEND
● STREAM FLOW GAUGE
○ WATER LEVEL GAUGE

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FLOOD FORECASTING AGREEMENT - PHASE ONE

BOYNE RIVER WATERSHED
HYDROMETRIC NETWORK

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
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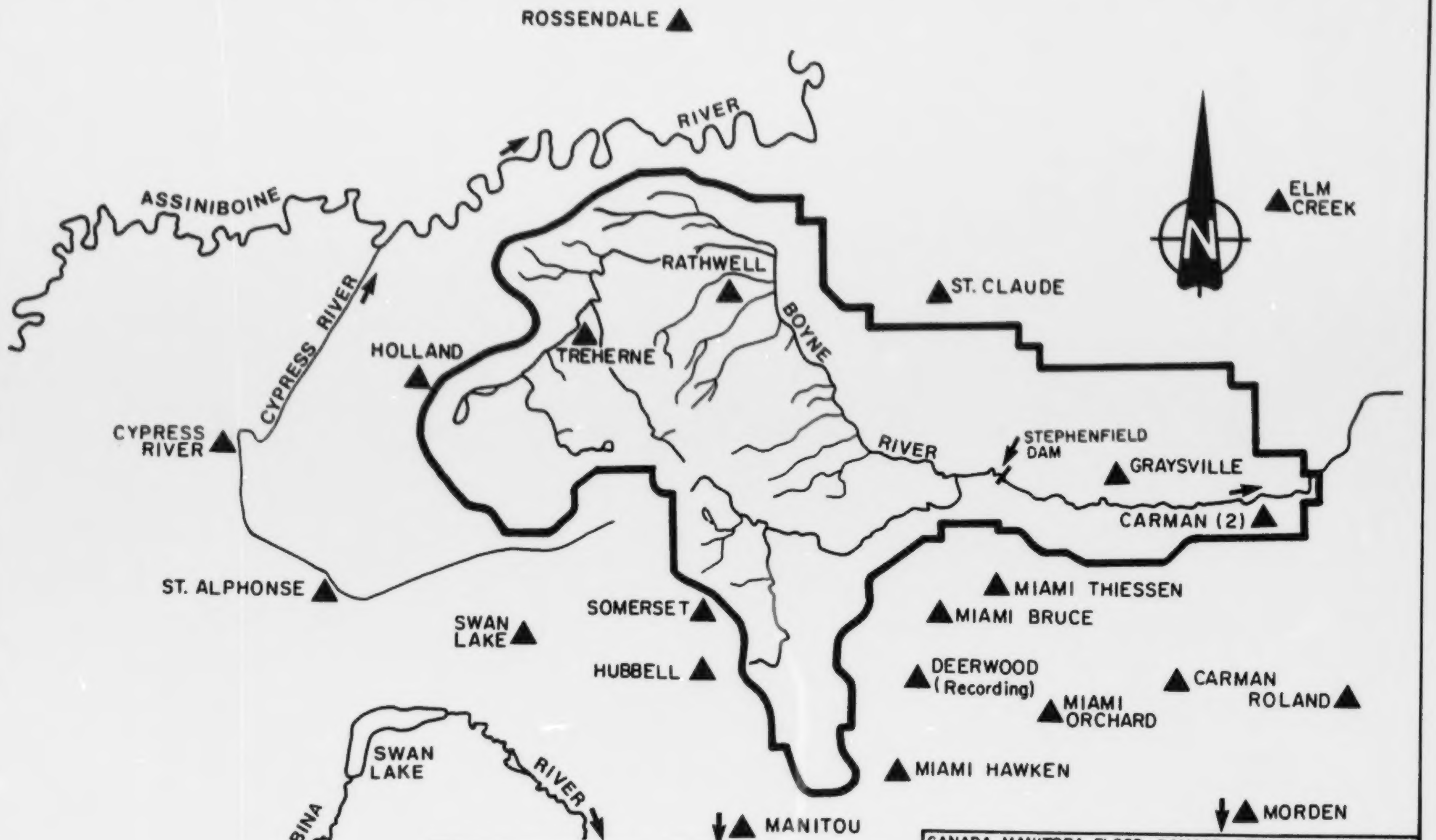
FIGURE C-7

FIGURE C-8



ROSSENDALE
ASSINIBOINE
RIVER
CYPRESS RIVER
CYPRESS RIVER
HOLLAND
TREHERNE
RATHWELL
BOYNE
RIVER
ST. CLAUDE
ELM CREEK
N
STEPHENFIELD DAM
GRAYSVILLE
CARMAN (2)
ST. ALPHONSE
SWAN LAKE
SOMERSET
HUBBELL
MIAMI THIESSEN
MIAMI BRUCE
DEERWOOD (Recording)
MIAMI ORCHARD
CARMAN
ROLAND
MIAMI HAWKEN
SWAN LAKE
RIVER
PEMBINA
ROCK LAKE
MANITOU
MORDEN

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
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CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## BOYNE RIVER WATERSHED
## CLIMATOLOGICAL NETWORK

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET OF | FILE NO |
|---|---|---|---|---|---|---|---|
| A.A.W. | CHECKED |  |  | AS SHOWN |  |  |  |

## Appendix D

# PREPARATION OF DATA INPUTS FOR HYDROLOGIC MODELS

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## D.1 INTRODUCTION

This appendix describes the procedures used to prepare and organize the climatological and hydrological data which was used for the calibration and verification of the hydrologic models used in the Pilot Project.

The hydrologic models employed in this study required an assortment of hourly, six-hourly, daily and seasonal data files for a variety of climatological elements as shown in Table D-1. In this report the six-hourly data is sometimes referred to as synoptic data. The synoptic average and synoptic total are the average and the total of the six-hourly records for the element in question for the periods 0100 to 0600, 0700 to 1200, 1300 to 1800, and 1900 to 2400, local standard time.

The climatological data files were purchased from the Atmospheric Environment Service of Canada (AES). Hydrometric data files were obtained from the Canadian Water Resources Branch (CWRB). Some supplementary data was also available from Manitoba Water Resources Branch (MWRB) files.

Data time series were prepared for the period from 1968 to 1979 since data for most of these years were required either for calibration or verification of the models.

## D.2 DATA FILE PREPARATION

The more complex hydrologic models used in the Pilot Project required continuous data files which represented areal averages of input data parameters for the watershed and/or the watershed segments. Since the data was not available in this form, considerable data analysis and manipulation was required to prepare the appropriate data time series in the form required by the models.

Point data at meteorological recording stations had to be converted to areal averages for the watersheds and into the appropriate time intervals coinciding with the computational time step employed in the models. Gaps in the time series of records, i.e. missing data, had to be filled with estimates based on data at nearby stations. Data errors at recording stations had to be corrected. For those data elements not recorded in the watershed, it was necessary to transfer data from outside data stations to the watershed.

Table D-2 lists the steps involved in preparing the input data time series. In essence, the procedure is to take whatever data is available in or near the watershed for a given element, estimate the missing values by interpolation or from data at nearby stations, and develop a representative watershed data file from the point sources of data. For many of the climatic elements which were required in the study no data was available in the watershed. Files for these elements had to be developed based totally on data recorded outside the Boyne River watershed. These outside sources are listed in Table D-3 and are located on Figure D-1.

A detailed description of how the steps outlined in Table D-2 were followed is given for each element in the following sections of the report.

Precipitation is the only element for which AES reports six-hourly or synoptic totals at its main meteorological network stations. For all other elements the synoptic files were developed from the hourly data files by summing or averaging the hourly records for the appropriate time periods.

One further complication arose because for two of the hydrologic models the Boyne River basin was divided into six segments as shown on Figure D-2, each of which was modelled separately. The multiple basin approach required that data files be prepared for each element for each of the six segments. Thus, upon conclusion, complete data files for each of the elements listed in Table D-1 were available for the Boyne River watershed as a whole, and for each of the six segments.

## D.3 SUMMER PRECIPITATION (APRIL TO OCTOBER)

Due to the high spatial variability of summer precipitation, it was desirable to use on-site data as much as possible to compute basin precipitation. At most climatological stations in and near the watershed precipitation is measured on a daily basis only. The station network is relatively sparse and varies from year-to-year. To illustrate network changes, daily precipitation networks for 1970 and for 1976 are shown on Figures D-3 and D-4.

Daily precipitation files were computed for the entire basin upstream of Stephenfield and also for the individual watershed segments used by the HSPF and SSARR models. Stations deemed to be most representative of each watershed segment, based on location, were grouped and arithmetic averages of the data at these stations were computed to obtain represent precipitation estimates. The station groupings used to compute daily segment precipitation are shown for each watershed segment in Table D-4. A similar procedure was used to compute daily precipitation for the entire watershed.

The HSPF model required hourly precipitation data for calibration. The nearest stations which record hourly precipitation data were at Deerwood and at Portage La Prairie. As shown on Figure D-1 Deerwood is located 7 km southeast of the Boyne River watershed, and Portage La Prairie is approximately 32 km north of the watershed. To compute hourly precipitation for a watershed segment, the computed daily segment precipitation was divided into 24 values on the basis of the hourly time distribution of precipitation at the closest hourly recording station.

## D.4 AMBIENT TEMPERATURE

Regression analysis between daily mean temperatures at selected high level and low level stations along the Manitoba Escarpment showed that the higher escarpment area normally has a temperature 1°C lower than the sub-escarpment area. Therefore, two temperature files were created for the watershed, one for the escarpment area and another for the sub-escarpment area. An hourly temperature file for the sub-escarpment area was produced using recorded hourly temperatures at Portage La Prairie, the nearest station with such data. The escarpment hourly temperature data was then produced by simply subtracting 1°C from all the values in the former data file. Escarpment temperatures were assigned to segments 1, 3 and 6, while sub-escarpment temperatures were assigned to other segments. For the single basin approach, the escarpment hourly temperatures were used since this area produces most of the basin runoff.

## D.5 DEW POINT TEMPERATURE

The only station near the watershed at which dew point temperatures are recorded is Portage La Prairie Airport. The method of computing watershed hourly dew point temperatures was identical to that for ambient temperatures, as described in Section D-4.

## D.6 WIND SPEED

The station closest to the watershed which records wind data is Portage La Prairie Airport. It was assumed that this wind data was representative for the entire watershed. Synoptic average winds were determined by computing the mean of the hourly wind speeds for the appropriate six-hour periods. The anemometer height at Portage La Prairie Airport is 10.06 metres. Since snowmelt equations employed in the HSPF and SSARR models require winds at the 15.24 metre level, the observed wind data was adjusted to this level by the commonly used power law formula (Buckler, 1969), as follows:

$$V_a = V_b \left(\frac{Ha}{Hb}\right)^{0.25}$$

where: $V_a$ = wind speed at elevation Ha
$V_b$ = wind speed at elevation Hb

## D.7 GLOBAL SOLAR RADIATION

Winnipeg Airport is the only station in southern Manitoba where global solar radiation is measured. It was considered unrealistic to assume that Winnipeg radiation data would be representative of that in the watershed because of the 80 km distance between the Winnipeg Airport and the watershed, and because of the probable effect of the escarpment on low cloud amounts. Therefore sunshine data from Morden, located close to the escarpment about 40 kilometres south of the watershed, was used. A relationship between global solar radiation and bright sunshine was first developed using data for Winnipeg Airport(Warkentin, pers. comm..). Hourly global radiation for Morden was then computed using this relationship and the bright sunshine data at Morden. The procedure is shown in Table D-5. Synoptic radiation data for Morden was computed by adding the hourly Morden radiation data for the appropriate time periods. The computed radiation data for Morden was assumed to be representative of radiation in the Boyne River watershed.

## D.8 POTENTIAL EVAPOTRANSPIRATION

The potential evapotranspiration (PE) data time series was computed on a daily time step only, based on observed pan evaporation data for Winnipeg Airport. It was assumed that the computed PE for the Winnipeg Airport would be representative for the Boyne River watershed. The pan evaporation data was first multiplied by the standard pan factor of 0.7 to provide a reasonable estimate of potential evapotranspiration from active vegetation (Kohler, et al 1955). It was then further multiplied by monthly factors to account for the typical type, density and state of vegetation through the seasons. For the SSARR and SLURP models these monthly factors were estimated using general information available in the literature (Gray, 1970), and a knowledge of the average vigour of local plant growth through the seasons. For the HSPF model, the monthly factors were based on values suggested in the ARMS user manual (Donigian and Davis, 1978). The monthly factors used were as follows:

| | May | June | July | August | September | October |
|---|---|---|---|---|---|---|
| SSARR, SLURP | 0.7 | 1.0 | 1.0 | 0.8 | 0.7 | 0.6 before frost<br>0.3 after frost |
| HSPF Surface Upper Zone | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| HSPF Lower Zone | 0.10 | 0.21 | 0.36 | 0.28 | 0.28 | 0.20 |

Actual daily evapotranspiration is computed internally in the hydrologic models, based on the input potential evapotranspiration and the computed soil moisture.

## D.9 SNOWPACK

The snowpack existing just prior to active spring snowmelt can be determined by direct snow course measurements, or may be estimated from recorded precipitation from freeze-up in the fall to break-up in the spring. The Manitoba Water Resources Branch measures snow water equivalents each February and March, but the data is too limited to be of much use in the Boyne River watershed. AES also measures snow depth at some stations but again the network is sparse. Therefore the end of winter snowpack was estimated by summing winter precipitation and then adjusting for estimated over-winter losses.

Gross precipitation totals were computed from subjective analyses of cumulative winter precipitation. A sample analysis for the winter of 1978-79 is shown on Figure D-5.

The snowpack estimates included some adjustments of reported snowfall water contents in 1970, 1971, and 1979. Heavy snowstorms occurred in March and April of these years. The AES Climate Station precipitation for these storms was computed assuming a snow density of 0.1. However the snow was unusually wet and heavy for these events, thereby containing more water per centimeter than the assumed density would indicate. Adjustments were therefore made, based on a relationship between the density of fresh snow and the air temperature at the time of the storm (Snow Hydrology, 1956).

The only winter losses considered were evaporation/sublimation losses. These were compiled using the Meyer formula (Meyer, 1942). Although the Meyer formula was not designed to estimate sublimation losses from snow, it should provide at least a rough approximation of the loss since both the loss and the Meyer equation evaporation are highly related to temperature. Fortunately in southern Manitoba winter losses are normally small compared to total winter precipitation, particularly in years when major spring runoff events occur. This obviates the requirement for accurate snow loss data. It was deemed that snowpack estimates would be improved by the subtraction of Meyer estimated losses, despite the lack of precision in these loss estimates.

Table D-6 summarizes the computed snowpacks for the winters preceding the springs of 1969, 1970, 1971, 1974, 1976 and 1979.

## D.10 LAST DATE OF SNOWMELT

The date when the snowpack was fully melted was estimated for each year under study to facilitate calibration of the snowmelt components of the models. Estimates were based on daily reported snow depths at Portage La Prairie Airport and on the time of the peak flow from snowmelt runoff in small nearby watersheds. Experience has shown that for a sustained rapid snowmelt, the peak flow from small (approximately 200 km$^2$ ) prairie watersheds occurs about the same time that the snowpack is melted (A. A. Warkentin, pers. comm.).

## D.11 STREAMFLOW DATA

Time series of mean daily discharges and stages for four stream gauging stations and one reservoir station in the Boyne River Watershed upstream of Stephenfield were obtained on magnetic tape from the Canada Water Resources Branch. These hydrometric data files were generally complete and required little further preparation. The stations are described in Appendix C-1.

## D.12 OBSERVATION

A great deal of the effort expended on the Pilot Study of the Boyne River Basin was directed to preparation of the input data files as described in this appendix. This was necessary to improve the quality of input data for the models, resulting in a more valid comparison of model capabilities and performance.

The large effort required in preparing the input files does raise the question of whether or not such models can be of practical use for operational flow forecasting. This matter is addressed in the main report.

## D.13 LITERATURE CITED

1. Buckler, S.J., 1969. The Vertical Wind Profile of Monthly Mean Winds Over the Prairies, TEC 718. Meteorological Service of Canada, Toronto, Ontario.

2. Donigian, A.S. Jr., Davis, H.H. Jr., 1978. User's Manual for Agricultural Runoff Management (ARM) Model. Environmental Research Laboratory, U.S. Environmental Protection Agency, Athens, Georgia. 30605.

3. Gray, D.M., 1970. Principles of Hydrology. College of Engineering, University of Saskatchewan, Saskatoon, Saskatchewan.

4. Kohler, M.A., Nordenson, T.J. and Fox W.E., 1955. Evaporation form Pans and Lakes. U.S. Weather Bureau Research, Paper 38.

5. Meyer, A.F., 1942. Evaporation from Lakes and Reservoirs, Minnesota Resources Commission, St. Paul, Minnesota.

6. Snow Hydrology, 1956. North Pacific Division Corps of Engineers, U.S. Army, Portland, Oregon.

### Table D-1
### Climatological Element Time Series Required for Hydrologic Model Calibration and Verification
### Period: 1968-1979

| Daily | Synoptic (6 Hours) | Hourly | Seasonal |
|---|---|---|---|
| Mean Ambient Temperature | *Ambient Temperature | Ambient Temperature | ***Net Winter Precipitation (November 1 to March 31) |
| | *Dew Point Temperature | Dew Point Temperature | |
| Total Precipitation | Precipitation | Precipitation | |
| | *Wind Speed | Wind Speed | |
| | *Global Radiation | Global Radiation | |
| Total Potential Evapotranspiration | **Potential Evapotranspiration | **Potential Evapotranspiration | |

Notes:

* - Computed from hourly data. Synoptic values are the mean of the hourly values for the period except for radiation where it is the sum of the hourly values.

** - Computed from daily data. Synoptic values are obtained by simply dividing daily values into four equal parts.

*** - Computed from monthly and daily data. The net winter precipitation is the cumulative precipitation from freeze-up to spring break-up minus Meyer evaporation.

### Table D-2
### Steps in Preparation of Climatological Data Time Series

1. Acquire climatological data tapes from AES.
2. Make a sub-set file of data required for project.
3. Check files for errors and completeness of record. Make corrections and fill in missing data by linear interpolation of data encompassing the data error or gap, or by extrapolating data from nearby stations.
4. Determine what available point data sources are most representative for the watershed based on location, topography and past experience.
5. Devise methodologies to compute on-site data from remote data, e.g. computation of watershed radiation from Winnipeg radiation and Morden sunshine data.
6. Devise methodologies to compute watershed segment data from point data sources.
7. Compute data files for synoptic (six-hour) and daily time intervals using hourly data.
8. For each continuous hydrologic model, write a program to re-format data files into the specific format required by the model.

## Table D-3
## Data Sources for Climatological Elements

| Parameter | Data Source |
|---|---|
| Ambient Temperature | Portage Airport |
| Dew Point Temperature | Portage Airport |
| Wind Speed | Portage Airport |
| Precipitation (daily) | All climatological stations in vicinity of watershed. |
| Precipitation (synoptic and hourly) | Computed from daily basin precipitation file using time distribution information from Deerwood and Portage Airport. |
| Global Radiation | Computed from data at Winnipeg Airport, based on sunshine comparison between Winnipeg and Morden. |
| Potential Evapotranspiration | Winnipeg Airport Pan Evaporation. |
| Net Winter Precipitation | Isohyetal Analysis of total seasonal precipitation based on data from all climate stations in vicinity of watershed. Evaporation at Winnipeg, based on Meyer method, subtracted. |

### Table D-4
### Climatological Station Groupings Used to Compute Segment Precipitation

| SEGMENT1 | SEGMENT 2 |
|---|---|
| Cypress River | Miami Thiessen |
| Rathwell | Rathwell |
| Somerset | St. Claude |
| St. Alphonse | |
| Treherne | |
| **SEGMENT 2 & 3** | **SEGMENT 6** |
| Rathwell | Deerwood |
| St. Claude | Miami Hawken |
| Treherne | Somerset |
| **SEGMENT 4** | **SEGMENT 7** |
| Deerwood | Carman 2 |
| Rathwell | Graysville |
| Somerset | |
| Treherne | |

Notes:

Selection of stations for specific segments was based on the location of the stations with respect to the segments and on the difference in elevation between the stations and the land segment.

## Table D-5
## Computation of Watershed Global Radiation Using Sunshine Data at Morden and Winnipeg

$$\mathrm{BWCR} = \mathrm{WOR} + C(\mathrm{MOS}-\mathrm{WOS})$$

Where:
BWCR = Boyne watershed hourly computed global solar radiation (megajoules/$m^2$)
WOR = Winnipeg hourly observed global solar radiation (megajoules/$m^2$)
MOS = Morden hourly observed bright sunshine in tenths of an hour (values range from 0 to 10)
WOS = Winnipeg hourly observed bright sunshine in tenths of an hour (values range from 0 to 10)
C = Factor relating bright sunshine hours to direct solar radiation. This is variable with the time of the day and the season as follows:

| Hour Ending | C | Hour Ending | C |
|---|---|---|---|
| 0100-0600 c.s.t. | 0.000 | 1300 | 0.135 |
| 0700 | 0.020 | 1400 | 0.130 |
| 0800 | 0.050 | 1500 | 0.110 |
| 0900 | 0.085 | 1600 | 0.080 |
| 1000 | 0.115 | 1700 | 0.050 |
| 1100 | 0.130 | 1800 | 0.020 |
| 1200 | 0.135 | 1900-2400 | 0.000 |

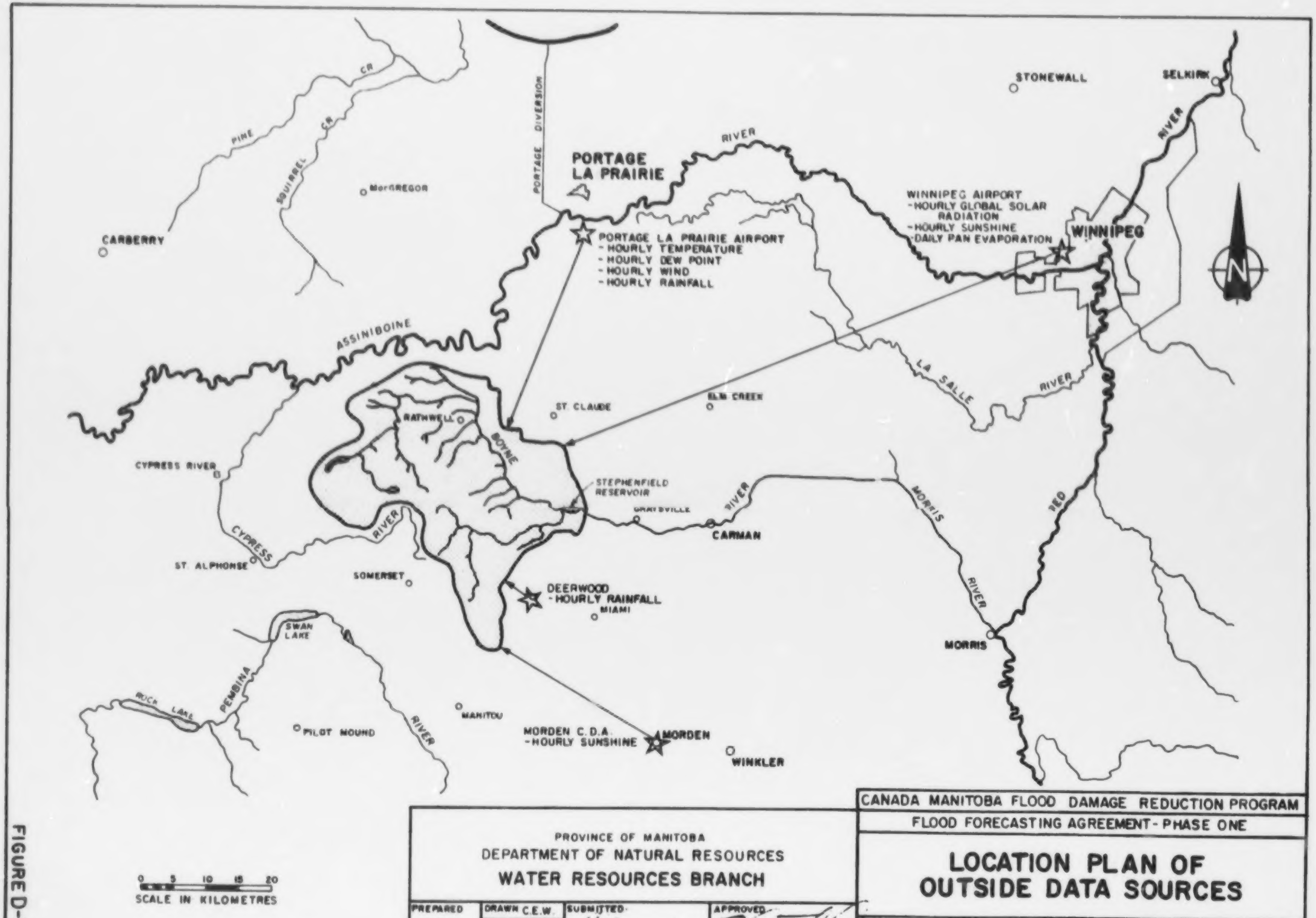
Seasonal Factors for C:
March 15 - 23 = 0.85
March 24 - 31 = 0.95
April 1 - 10 = 1.05
April 11 - 22 = 1.15

## Table D-6
## Net Winter Snowpack Estimates for Calibration and Verification Years (based on Isohyetal Analyses of Cumulative Winter Precipitation)

| | Data in Millimetres of Water | | | | | |
|---|---|---|---|---|---|---|
| | Nov. 1/68-Mar. 31/69 | Nov. 1/69-Mar. 31/70 | Nov. 1/70-Mar. 31/71 | Nov. 1/73-Mar. 31/74 | Nov. 1/75-Mar. 15/76 | Nov. 1/78-Mar. 31/79 |
| Segment 1 | 128 | 165 | 155 | 145 | 164 | 184 |
| Segment 2 | 122 | 165 | 145 | 130 | 164 | 186 |
| Segment 3 | 117 | 165 | 150 | 135 | 164 | 180 |
| Segment 4 | 127 | 165 | 155 | 155 | 164 | 180 |
| Segment 5 | 120 | 165 | 150 | 150 | 164 | 186 |
| Segment 6 | 128 | 165 | 150 | 155 | 164 | 169 |
| Entire Basin U/S Stephenfield | 125 | 165 | 153 | 149 | 164 | 180 |
| Notes: | | | | | | |
| Evaporation | 35 | 28 | 34 | 25 | 36 | 31 |
| Adjustment for estimated error in reported water content of April snowfall | 0 | +50 | +41 | 0 | 0 | +15 |

Notes:

The end dates for the winter periods specified above, i.e. March 15 or March 31, do not signal the precise end of winter. Since the spring simulation period began March 16 or April 1, significant additional snowfall was in some years input through the daily precipitation input time series.


STONEWALL
SELKIRK
RIVER
PORTAGE LA PRAIRIE
PORTAGE DIVERSION
PINE
CR
SQUIRREL CR
MacGREGOR
CARBERRY
PORTAGE LA PRAIRIE AIRPORT
- HOURLY TEMPERATURE
- HOURLY DEW POINT
- HOURLY WIND
- HOURLY RAINFALL
WINNIPEG AIRPORT
- HOURLY GLOBAL SOLAR RADIATION
- HOURLY SUNSHINE
- DAILY PAN EVAPORATION
WINNIPEG
N
ASSINIBOINE
LA SALLE RIVER
RATHWELL
BOYNE
ST. CLAUDE
ELM CREEK
CYPRESS RIVER
STEPHENFIELD RESERVOIR
GRAYSVILLE
RIVER
CARMAN
MORRIS RIVER
RED
CYPRESS RIVER
ST. ALPHONSE
SOMERSET
DEERWOOD
- HOURLY RAINFALL
MIAMI
MORRIS
SWAN LAKE
PEMBINA
ROCK LAKE
RIVER
MANITOU
PILOT MOUND
MORDEN C.D.A.
- HOURLY SUNSHINE
MORDEN
WINKLER
0 5 10 15 20
SCALE IN KILOMETRES

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

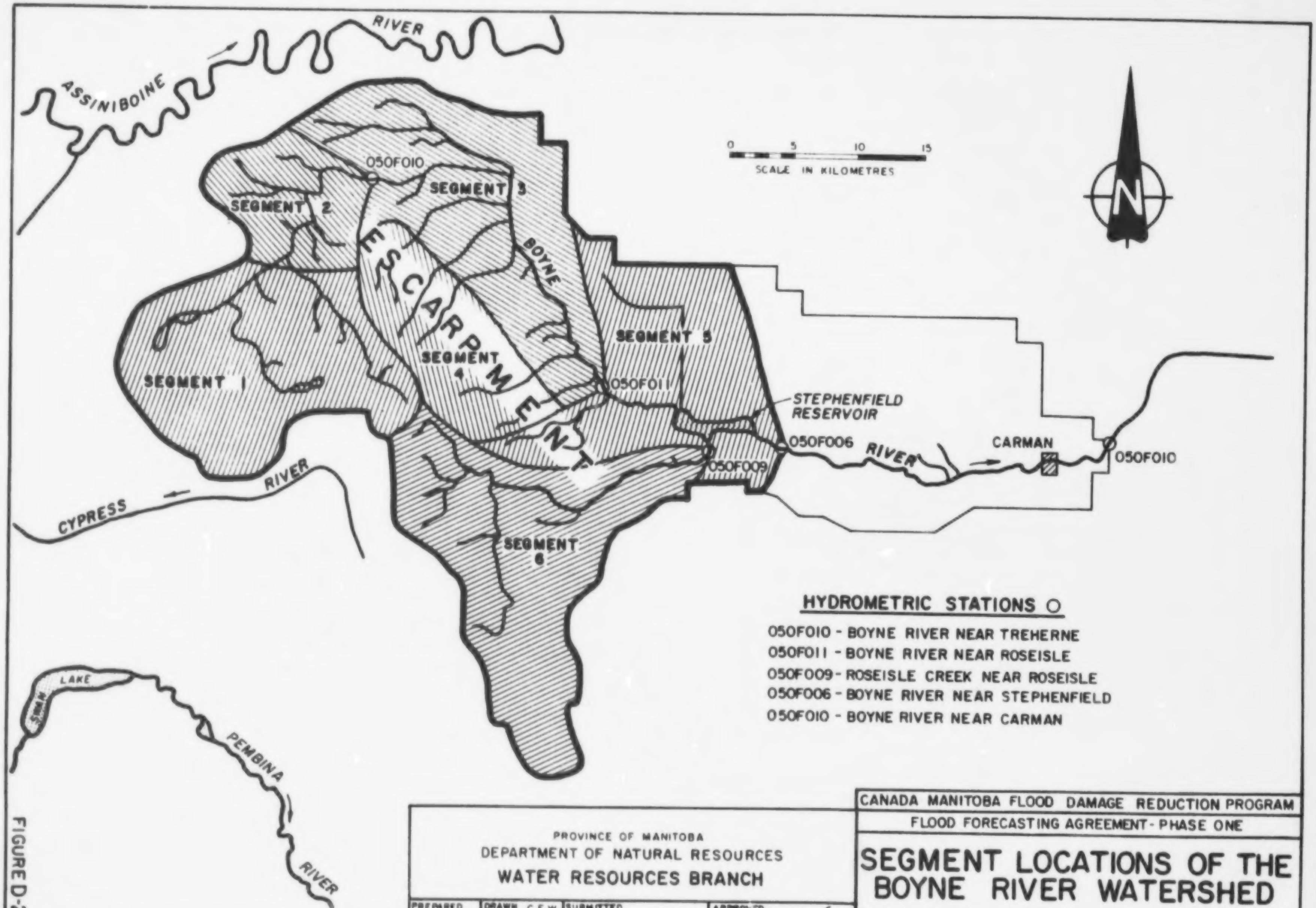
| PREPARED | DRAWN C.E.W. | SUBMITTED | APPROVED |
|---|---|---|---|
| A.A.W. | CHECKED | | |

CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## LOCATION PLAN OF OUTSIDE DATA SOURCES

| SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|
| AS SHOWN | | OF | |

FIGURE D-1


ASSINIBOINE RIVER
CYPRESS RIVER
PEMBINA RIVER
SWAN LAKE
BOYNE RIVER
ESCARPMENT
SEGMENT 1
SEGMENT 2
SEGMENT 3
SEGMENT 4
SEGMENT 5
SEGMENT 6
050F010
050F011
050F009
050F006
STEPHENFIELD RESERVOIR
CARMAN
0 5 10 15
SCALE IN KILOMETRES
N

**HYDROMETRIC STATIONS ○**

050F010 - BOYNE RIVER NEAR TREHERNE
050F011 - BOYNE RIVER NEAR ROSEISLE
050F009 - ROSEISLE CREEK NEAR ROSEISLE
050F006 - BOYNE RIVER NEAR STEPHENFIELD
050F010 - BOYNE RIVER NEAR CARMAN

CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## SEGMENT LOCATIONS OF THE BOYNE RIVER WATERSHED

PROVINCE OF MANITOBA
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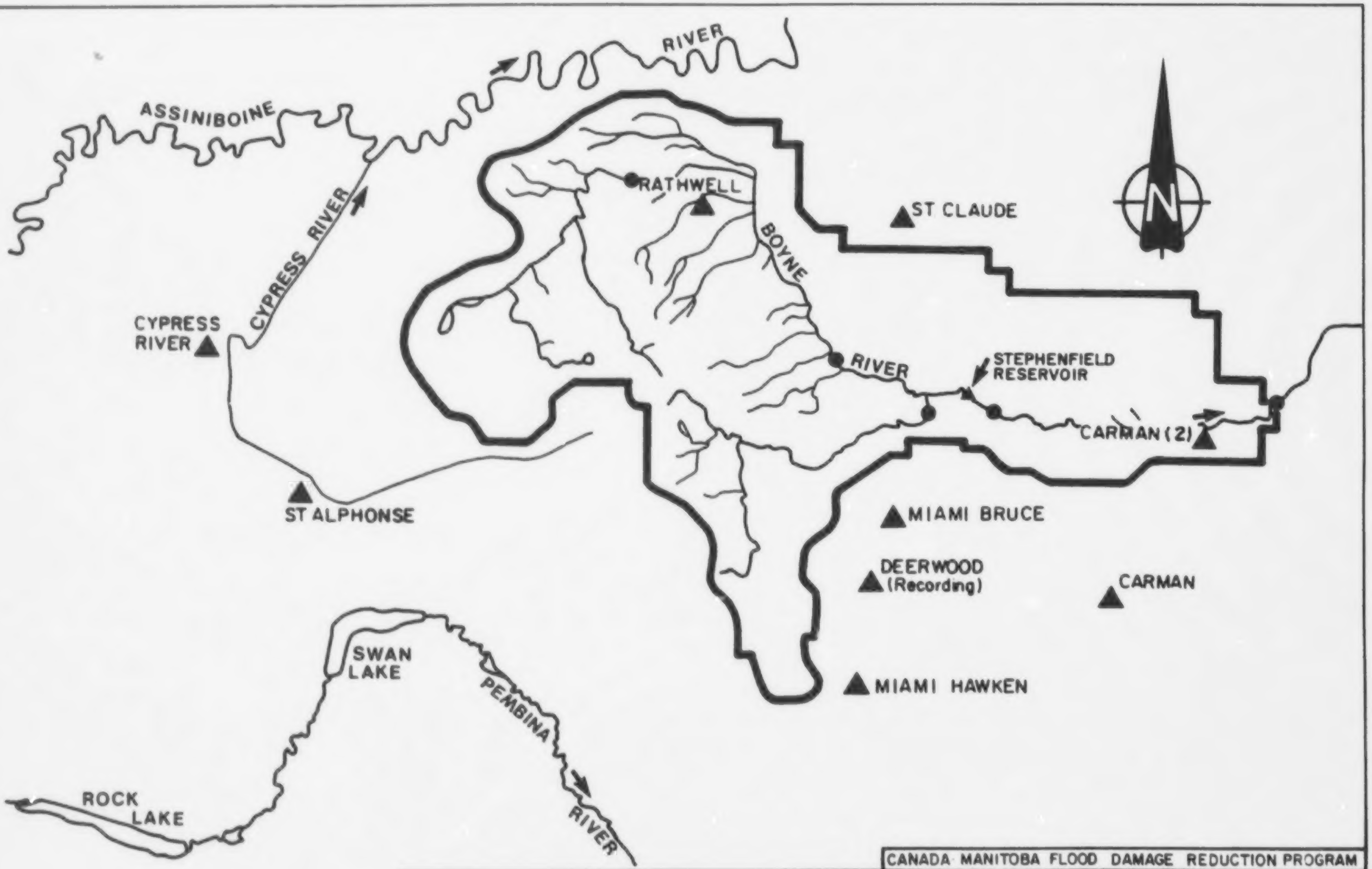
WATER RESOURCES BRANCH

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO |
|---|---|---|---|---|---|---|---|
| G E M | C.E.W.<br>CHECKED A.W. | | | AS SHOWN | | OF | |

FIGURE D-2


ASSINIBOINE
RIVER
CYPRESS RIVER
CYPRESS RIVER
ST ALPHONSE
RATHWELL
BOYNE
RIVER
ST CLAUDE
STEPHENFIELD RESERVOIR
CARMAN (2)
MIAMI BRUCE
DEERWOOD (Recording)
CARMAN
MIAMI HAWKEN
SWAN LAKE
ROCK LAKE
PEMBINA
RIVER
N

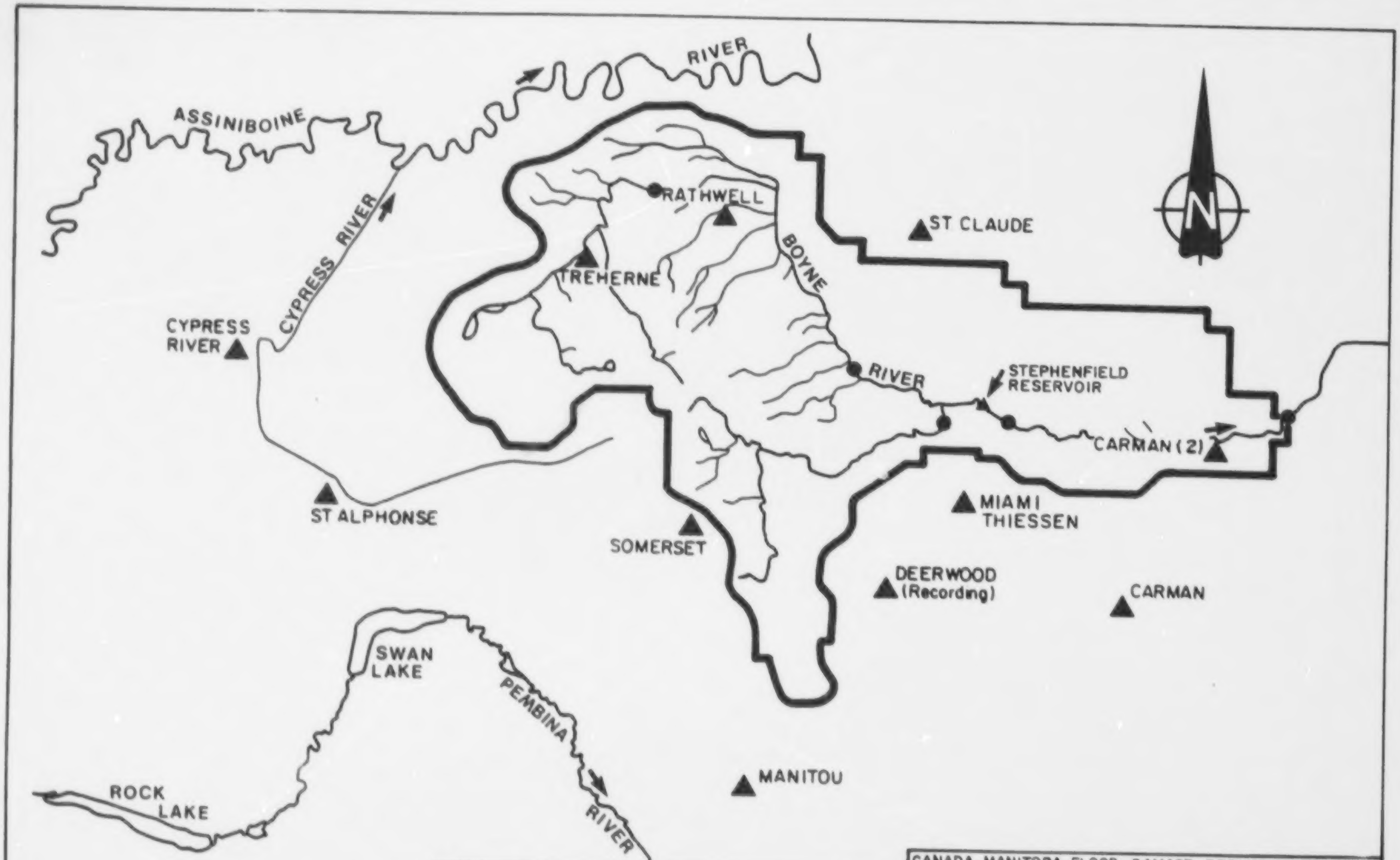
PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

BOYNE RIVER WATERSHED
DAILY PRECIPITATION NETWORK
1970

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO |
|---|---|---|---|---|---|---|---|
| A. A. W. | CHECKED | | | AS SHOWN | | OF | |

FIGURE D-3


N
ASSINIBOINE
RIVER
CYPRESS RIVER
CYPRESS RIVER
ST ALPHONSE
RATHWELL
TREHERNE
BOYNE
RIVER
ST CLAUDE
STEPHENFIELD RESERVOIR
CARMAN (2)
MIAMI THIESSEN
SOMERSET
DEERWOOD (Recording)
CARMAN
SWAN LAKE
PEMBINA
RIVER
ROCK LAKE
MANITOU

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FLOOD FORECASTING AGREEMENT - PHASE ONE

BOYNE RIVER WATERSHED
## DAILY PRECIPATION NETWORK
## 1976

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO |
|---|---|---|---|---|---|---|---|
| A. A. W. | CHECKED |  |  | AS SHOWN |  | OF |  |

FIGURE D-4

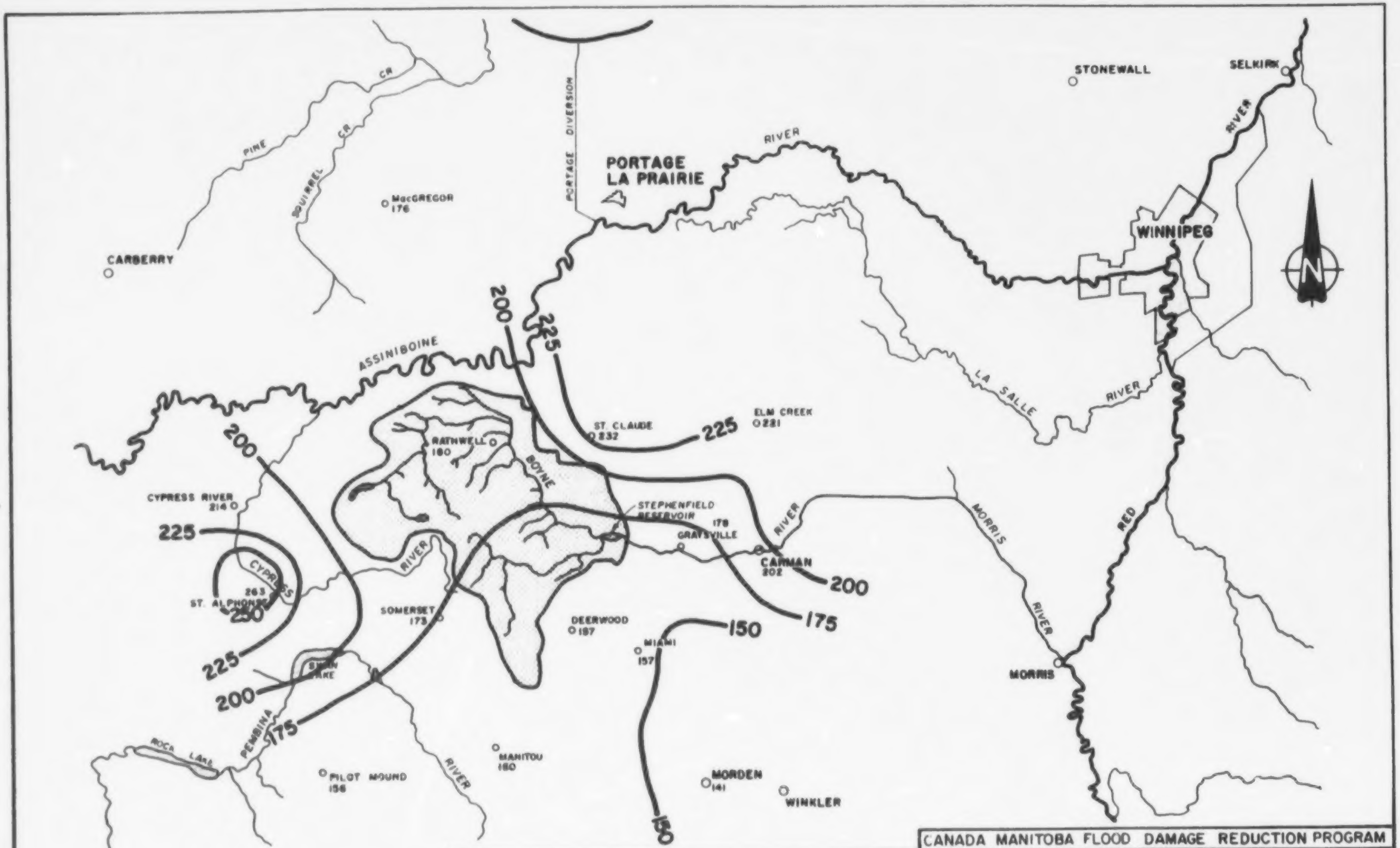
SELKIRK
STONEWALL
RIVER
WINNIPEG
PORTAGE LA PRAIRIE
PORTAGE DIVERSION
PINE
SQUIRREL CR.
CR.
MacGREGOR 176
CARBERRY
ASSINIBOINE
200
225
ST. CLAUDE 232
225
ELM CREEK 221
LA SALLE
RIVER
RATHWELL 180
BOYNE
STEPHENFIELD RESERVOIR
179
GRAYSVILLE
RIVER
CARMAN 202
200
175
MORRIS
RIVER
RED
MORRIS
CYPRESS RIVER 214
225
CYPRESS
263
ST. ALPHONSE
250
225
200
RIVER
SOMERSET 173
DEERWOOD 197
MIAMI 157
150
150
SWAN LAKE
175
PEMBINA
ROCK LAKE
PILOT MOUND 156
MANITOU 180
RIVER
MORDEN 141
WINKLER
N
0 5 10 15 20
SCALE IN KILOMETRES

FIGURE D-5

PROVINCE OF MANITOBA
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WATER RESOURCES BRANCH

CANADA MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
FLOOD FORECASTING AGREEMENT - PHASE ONE

## ISOHYETAL MAP OF GROSS WINTER PRECIPITATION (mm)

NOVEMBER 1, 1978 – MARCH 31, 1979

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO |
|---|---|---|---|---|---|---|---|
| A.A.W. | C.E.W. |  |  | AS SHOWN |  | OF |  |
|  | CHECKED |  |  |  |  |  |  |

## Appendix E

# CALIBRATION OF HYDROLOGIC MODELS

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## E.1 INTRODUCTION

This appendix describes the methods used to calibrate the three models selected for detailed study in the Pilot Project. The three models are described in Appendix D.

A complete hydrologic model must be capable of simulating snowmelt, movement of the input moisture from the point of application to a receiving stream, and movement of the water down that stream. These three processes are often referred to as the snowmelt phase, the lands phase, and the conveyance phase. This appendix concentrates on the first two phases. The conveyance or channel routing phase is discussed in Appendix F.

The spring runoff events on the Boyne River which were selected for calibration were those of 1970, 1976 and 1979. These events constituted a minor flood, a within - bank condition and a major flood respectively.

Model simulated flows for the calibration years were compared to those of the presently used MANAPI forecasting procedure.

The calibration runoff models for snowmelt induced floods involves somewhat different methodologies than that for rainfall induced floods. Rainfall induced floods are rare in Manitoba and snowmelt runoff produces most flood damages in the Red, Assiniboine, and Souris River basins. Therefore the model calibration efforts were concentrated on producing accurate simulations of spring runoff events caused mainly by snowmelt.

The models were not run on data for the winter period for several reasons. Firstly winter losses from the snowpack are typically negligible due to the absence of above freezing temperatures and the usual lack of sufficient energy for sublimation. Losses may on occasion be significant during unusually mild winters, but such winters are usually dry and therefore are of little concern with respect to flood forecasting. Secondly it was thought that good estimates of the snowpack could be obtained from subjective isohyetal analyses of cumulative winter precipitation and, therefore, the time and expense of running the snow accumulation and ablation subroutine could be avoided. Besides, of the three models tested, only HSPF has the facility to simulate winter snow ablation.

## E.2 SEGMENTED WATERSHED APPROACH

### E.2.1 Watershed Discretization

With the SSARR and HSPF models, calibrations were performed using both the single basin and the segmented basin approaches. In the single basin approach, a single set of data inputs and model parameters were used for the entire watershed upstream of Stephenfield. In the segmented or multiple basin approach the Boyne River watershed was divided into six unique areas or segments, based on physiographic watershed features and the location of hydrometric stations, as described in Section 2.6

of Appendix C. A large number of segments was originally envisaged to accommodate the great areal variability in topography and soil types within the watershed, but this was reduced to six in order to reduce the time and cost of calibration. These segment locations are shown on Figure E-1.

Most segments were delineated so that their downstream points were located at hydrometric stations. This was done to facilitate the computation of segment flows required for model calibration.

Of the six segments within the Boyne River watershed upstream of Stephenfield, particular emphasis was placed on calibrating Segment 6, Roseisle Creek near Roseisle. This segment was chosen for an independent comparison of model performance for a smaller, relatively homogeneous watershed where the complicating factor of channel routing could be avoided without serious consequences.

### E.2.2 Computation of Segment Flows for Calibration

Segment 6 was the only segment where observed segment flows were available for model calibration. For all the other segments it was necessary to compute segment flows. The method of computation varied from segment to segment depending on the availability of hydrometric data and the significance of travel time and attenuation on the mainstream.

It was not possible to compute flows for Segment 1 and for Segment 2 because there is no hydrometric station at the boundary between these two segments. As a result, these two segments were combined into a single segment for SSARR model calibrations. The recorded flows at station 050F010 at the end of this larger combined segment were used for SSARR model calibration. With the HSPF model an attempt was made to calibrate Segments 1 and 2 individually using only the recorded streamflow data at the outlet of Segment 2 as a check.

Flows for Segments 3 and 4 were likewise not directly available for model calibration. Daily flows for Segments 3 and 4 combined were computed by routing flows for the Boyne River near Treherne (station 050F010) to the Boyne River near Roseisle (station 050F011) and by subtracting these routed flows from the recorded flows at the latter station. The Muskingum-Cunge method of channel routing was used. This method of routing is briefly described in Appendix F.

Runoff from Segment 5 was computed by summing the recorded flows for the Boyne River near Roseisle and Roseisle Creek near Roseisle, routing the flows through the Stephenfield Reservoir, and subtracting the resulting flows from the recorded flows on the Boyne River near Stephenfield. Channel routing of flows was not deemed necessary due to the relatively short distances between these stations and the lack of overbank flows.

## E.3 MODEL CALIBRATION WITH SUMMER DATA

### E.3.1 Introduction

Although the goal of this study was to simulate snowmelt runoff conditions, the models had to be run for the preceding summer and fall to provide an estimate of soil moisture condition at freeze-up. Therefore the emphasis of the summer calibrations was on obtaining a good approximation of the runoff volume for rainstorm events and for the summer season as a whole. It was surmised that this was the main requirement which would make the computed fall basin moisture storages plausible. There was no soil moisture data available for comparison. Little emphasis was placed on timing, i.e. on matching simulated and observed hydrographs for individual runoff events. The one exception to this was the HSPF model where timing was important to the determination of appropriate model parameters.

In the HSPF model many of the process parameters involved in the snowmelt routine and in the lands phase must be calibrated. To simplify the procedure it is preferable to calibrate the parameters associated with the lands phase first, and to then address the calibration of the snowmelt processes. This can be achieved by calibrating the soil-related parameters on summer runoff events where snowmelt is not involved. The resulting parameters can then be used in the spring, where the calibration effort can concentrate on the snowmelt processes.

Initially all models were run using a twenty-four hour time step for the summer period. However, the time step for the HSPF model was later changed to an hour since a more accurate representation of precipitation input was required to effectively calibrate the model.

The following sections describe how each of the three models were calibrated with summer data.

### E.3.2 Summer Calibration - HSPF Model

#### E.3.2.1 General Approach

A daily time interval was initially used for calibration and soil moisture computations. However, in the HSPF model, infiltration is highly dependent on the rainfall intensity. Use of daily rainfall data led to a gross underestimate of rainfall intensity for most summer storms. This resulted in the simulation of too little runoff and too much soil moisture. Calibration was therefore changed to the use of an hourly time period with hourly precipitation input. Hourly potential evapotranspiration inputs were obtained by dividing daily values into 24 equal portions.

It had originally been planned to calibrate each of the six segments under the segmented basin approach. After some experience in calibrating the model a shortcut procedure was devised where only Segments 5 and 6 were calibrated in detail. These two segments represent the two basic soil types throughout the watershed; soils with low infiltration rates (Segments 1, 3, 4 and 6) and soils with high infiltration rates (Segments 2 and 5). The model process parameters and fall soil moisture obtained from the Segment 5 calibrations were later applied to Segment 2 and those obtained for Segment 6 were applied to Segments 1, 3 and 4.

The unique physical features of each segment, such as slope and the length of the overland flow plane, were still retained for use in simulating spring flows from individual segments.

For Segment 6, recorded flows on Roseisle Creek were used as a basis for calibration. For Segment 5, the simulated flows were added to recorded upstream flows entering this segment and the total flows were routed through the Stephenfield Reservoir. Routed outflows from the Reservoir were then compared with recorded flows on the Boyne River just downstream of the Reservoir.

### E.3.2.2 Calibration Techniques

Most of the important physical processes from snowmelt through to flow routing are modelled within HSPF. The model is described in detail in Appendix D.

Calibration of the HSPF model for rainfall runoff simulation requires the determination of appropriate values for 20 parameters. These parameters are listed in Table E-1. The 20 parameters consist of 6 physical parameters and 14 process parameters. Physical parameters are those parameters that can be readily measured, like the slope of a land segment. Process parameters quantify the storage and movement of water in and on the land surface; they are difficult or impractical to measure and are, therefore, determined through model calibration.

Of the 14 process parameters, seven were set to their default values, since they were not critical and did not vary much from basin to basin. Techniques of estimating the remaining seven process parameters and the six physical parameters were obtained from the ARMS User's Manual (Donigian and Davis, 1978) and the Stanford IV Watershed manual (Crawford and Linsley, 1966). There was a slight problem in using these references, as the variable names are not the same in the two manuals. In Table E-1 the corresponding Stanford name is given in brackets next to the HSPF name.

The parameter list was further reduced by assigning initial estimates to five of the physical parameters, leaving the following eight parameters to be calibrated:

Interception Storage Capacity - CEPSC
Infiltration Rate - INFILT
Upper Zone Nominal Storage - UZSN
Lower Zone Nominal Storage - LZSN
Interflow Inflow Parameter - INTFW
Baseflow Evapotranspiration - BASETP
Groundwater Evapotranspiration - AGWETP
Lower Zone Evapotranspiration - LZETP

Determination of appropriate values for these eight parameters was done systematically. Firstly the parameters were adjusted to achieve a plausible seasonal water balance and to simulate the correct runoff volume for the period from May 1 to October 31 of the summer calibration years. This was done mainly by changing the size of the lower zone storage and the transpiration losses from it. Transpiration losses were varied by changing the monthly lower zone evapotranspiration factors (LZETP) which are used to adjust potential evapotranspiration based on the state of plant growth. The seasonal runoff could be reduced by increasing the size of the lower zone storage (LZSN) thereby increasing the transpiration from this zone or by increasing the monthly evapotranspiration factors. The infiltration parameter (INFILT) was also adjusted to affect the summer water balance. A higher infiltration rate allows more water to percolate into the ground and to be available for evapotranspiration thus reducing runoff.

Secondly, the model was calibrated to reproduce runoff from individual rainstorm events. Particular emphasis was placed on achieving a good simulation of flows for a major multiple rainstorm event which occurred in late June and early July, 1969.

This was done primarily by adjusting the upper zone nominal storage (UZSN), the interflow parameter (INTFW) and the infiltration parameter (INFILT). The upper zone storage affects the initial response a watershed has to a rainstorm, and its normal value depends on the slope of the basin and the amount of depressional storage in the basin. If the value is increased, more water is required to fill this zone before overland flow and interflow can develop. Adjusting this parameter therefore changed the rising limb of the simulated hydrograph. Increasing the interflow parameter (INTFW) resulted in more water going into interflow and less into overland flow. Since interflow is much slower than overland flow, this parameter affects the timing of runoff. This parameter was used to adjust the peak of the hydrograph. Minor adjustments to infiltration (INFILT) were found to have effects similar to adjustments in the interflow parameter (INTFW). Only minor adjustments, if any, were made to the infiltration rate in calibrating individual storm events so as not to significantly affect the summer water balance. Adjustments in interception storage capacity (CEPSC) resulted in varied initial losses of water to fill this storage. Adjustment of this parameter had relatively minor effects since interception storage was small.

Low flows, while not a primary concern, were adjusted by changing active groundwater evapotranspiration (AGWETP), and by changing the riparian (riverbank) evapotranspiration (BASETP). Changing these factors affected the summer water balance.

Since the model does not allow for monthly factors to account for leaf growth of plants in these zones, these parameters were changed during the summer by using the special action block of the model, as described in the HSPF User's Manual (Johanson et al, 1981).

## E.3.3 Summer Calibration - SSARR Model

### E.3.3.1 General Approach

As stated in the introduction, the purpose of calibrating the SSARR model on data for the summer months of 1969, 1975 and 1978 was to determine basin moisture storages at freeze-up in the late fall. These storages were required as starting conditions for the spring runoff calibration. Unlike the procedure with the HSPF model, the SSARR model summer calibrations were not used to determine model parameters for spring runoff simulation. This is due to basic differences in the models; the SSARR model employs empirical indices while the HSPF model is more physically-based.

SSARR model calibration involved use of the same basin segments employed for the HSPF model, with a few notable exceptions. Segments 1 and 2 were combined into a single segment hereafter referred to as Segment 1 & 2, and segments 3 and 4 were combined into a single segment hereafter referred to as Segment 3 & 4. These segments were combined to reduce the complexity of calibrations, particularly since recorded flow data were not available for the individual segments. In order to further reduce the workload, only Segment 1 & 2, Segment 6, and the entire watershed were calibrated on summer data. The fall moisture storages for Segment 3 & 4 and Segment 5 were interpolated from the results of the calibrated segments and a consideration of differences in topography and soil type.

### E.3.3.2 Calibration Techniques

The SSARR model uses empirical indices and relationships to simulate the various processes involved in runoff. A flowchart of the SSARR processes is given in Figure B-4.

SSARR model summer calibrations were performed using daily inputs for precipitation and potential evapotranspiration (PE). PE data included factors to account for the state of plant growth and the occurrence of killing frost, as described in Appendix D.

Calibration of the model on summer data involved, in the main, the construction of initial Runoff Percent and Soil Drying curves. These curves were then adjusted through a trial and error process until the difference between simulated and recorded runoff volumes was minimized.

Starting values of the Soil Moisture Index (SMI) for each summer run were estimated from data on the preceeding spring snowmelt recharge and on subsequent precipitation prior to May 1. It was found that rough estimates of the starting SMI were sufficient since small errors would not affect the simulated flows for more than a week or two.

Under the single basin approach, one set of Runoff Percent and Soil Drying curves was used for all the segments, and the lands conveyance parameters were common to all segments. The Final Runoff Percent and Soil Drying Curves obtained through calibration are shown on Figure E-2.

It is apparent from the Runoff Percent curves on Figure E-2 that Segment 6 (Roseisle Creek) produces higher runoff coefficients than the other segments. This is due to the predominance of fine textured clay loams in this sub-basin. These loams have a low infiltration capacity. Also, the escarpmental nature of the lower portion of this segment would tend to produce more runoff. The Runoff Percent curve for Segment 1 & 2 is somewhat lower than for Segment 6 for a given SMI, likely due to a somewhat lesser slope and more surface depressional storage. The Runoff Percent Curve for the entire Boyne River Watershed at Stephenfield is somewhat lower than on Segment 1 & 2 and on Segment 6 for a given SMI, indicating that on the average some other portions of the watershed do not produce as much runoff as the calibrated segments.

The Soil Drying curves were related to the type of soil in the basin and to the respective runoff curves. The general slope of the curve was estimated using information available in the literature and on hydrologic experience. This resulted in adequate initial estimates of these curves.

With respect to the curves on Figure E-2 it should be noted that in the range of soil moisture indices greater than 4.0 inches, the evapotranspiration was estimated to be close to or at 100 percent of the potential evapotranspiration. Under these conditions the watershed would be nearly saturated, with water standing in puddles and potholes. As the soil moisture index decreases to values below about 4.0 inches, the ratio of actual to potential evapotranspiration decreases rapidly and runoff coefficients become very small.

A rough estimate of the Baseflow Percent Curve was made based on an examination of summer flow hydrographs. This relationship is shown on Figure E-3. Only rough estimates of the Surface/Subsurface distribution curve and the routing parameters for surface, subsurface and baseflow were made since the emphasis for the summer calibrations was on simulating the correct runoff volume rather than the exact time distribution of runoff. A more concerted effort was made to define these parameters on Roseisle Creek in an attempt to simulate flows from the major rainstorms of late June and early July, 1969.

### E.3.4 Summer Calibration - SLURP Model

#### E.3.4.1 General Approach

Only the single basin approach was used in calibrating the SLURP model on the basin upstream of Stephenfield, since the model lacks the capability of handling multiple segments. Calibrations were also done on Roseisle Creek (Segment 6) to see how the model would perform on a smaller watershed. The model was calibrated using daily input data on temperature, precipitation and potential evapotranspiration. As with the SSARR model, the emphasis in SLURP summer calibrations was to obtain reasonably accurate simulations of runoff volume. Little effort was made to reproduce the shape of runoff hydrographs.

#### E.3.4.2 Calibration Techniques

The SLURP model is described in Appendix B. Figure B-9 shows the basic configurations of the SLURP model.

Since the timing factors for summer runoff were not emphasized, calibration of the SLURP model on summer data was reduced to the determination of appropriate values for five important variables. These are shown in Table E-2 together with the values obtained for the calibration years on the basin upstream of Stephenfield.

The calibration parameters were obtained through a trial and error process and by employing the optimization option within the model. Optimization runs were done by fixing some of the better known parameters and optimizing the rest. Between two and five parameters were optimized in any given run. Optimization runs were discontinued when it was determined that this procedure was producing unrealistic parameter estimates.

## E.4 MODEL CALIBRATION WITH SPRING DATA

### E.4.1 Introduction

The HSPF and SSARR models were run on a six-hour time interval for the spring runoff period. Selection of the six-hour interval for the spring was partly based on a comparison of simulated snowmelt for a six-hour and 6 one-hour periods. There was no significant difference in snowmelt and therefore the larger time step was selected to reduce costs. A twenty-four hour time interval was considered but was rejected because averaging the input parameters over twenty-four hours would eliminate the cyclical (diurnal) nature of some of the input parameters like temperature and radiation. Also, considering that the snowmelt period may extend for only three to five days, a daily time step would be too coarse. The SLURP model was not run on hourly data because it is designed to use daily data only.

Calibration of the models on spring data for 1970, 1976, and 1979 involved firstly the calibration of the snowmelt component of the models so as to achieve the correct date for the completion of snowmelt. Thereafter, lands phase and conveyance parameters were adjusted in order to achieve the appropriate runoff volume and time distribution of runoff. Evaporation was assumed to be zero during the active snowmelt runoff period, since pan evaporation was typically not available for this period. The latter is considered a reasonable assumption since the temperature of the meltwater is generally not higher than the dew point temperature of the air during the snowmelt runoff period. However, the water balance of the HSPF model suggested that evaporation was quite significant beginning a week or so after the peak from snowmelt runoff. Evaporation data was therefore used when available. Finally the simulated runoff was routed through the river system to a downstream measuring point.

Energy budget procedures were employed for snowmelt computations with HSPF and SSARR models. The SLURP and MANAPI models employ more simplistic melt factor and temperature index procedures. The SSARR model contains the option of using a degree-day/melt factor procedure instead of an energy budget. This method is attractive because it precludes the need for the extensive data files required by the energy budget procedure. Tests were done to compare snowmelt using the degree-day and energy budget procedures. The results showed that under most atmospheric conditions, the two methods produced similar results. However, for unusual atmospheric conditions, such as were experienced during the 1979 melt period, the performance of the energy budget procedure was much superior. The decision was therefore made to use the energy budget procedure where possible, i.e., for the HSPF and SSARR models.

The following sections describe the spring runoff calibration procedures for each of the three models.

## E.4.2 Spring Calibration - HSPF Model

### E.4.2.1 General Approach

The model process parameters for the lands phase, obtained from the summer calibrations, were used for spring runoff calibration. Values obtained for Segment 5 were also used for Segment 2 while values obtained for Segment 6 were also used for Segments 1, 3 and 4. The model parameters based on physical watershed data and the input data used for the various segments in the segmented basin approach were those which were unique to each segment. In the single basin approach, model process parameters obtained from summer calibrations on Segment 6 were used for the entire watershed as these were considered to be the most representative. The model parameters based on physical watershed data and the data inputs for the single basin approach were average values for the entire watershed. The fall soil and subsoil moisture storages for 1969, 1975 and 1978, obtained from the summer calibrations, were used as starting basin conditions for the spring calibrations.

Special emphasis was again placed on the calibration of Roseisle Creek (Segment 6) to enable the evaluation of the model on a smaller watershed where channel routing is of lesser importance.

Calibrations were performed with data averaged over six-hour time periods, as described in Appendix D, for the spring periods of 1970, 1976, and 1979. Data time series of representative ambient temperatures, dew point temperatures, precipitation amounts , solar radiation amounts, winds and streamflows were used. Evaporation data were used where available, i.e., when the calibration period extended into May. When pan evaporation data were unavailable, evapotranspiration was assumed to be zero.

Under both the single and segmented basin approaches, channel routing of flows was effected from the Boyne River near Treherne to the Boyne River near Roseisle and through the Stephenfield Reservoir.

### E.4.2.2 Calibration Techniques

Model process parameters for the lands phase of the HSPF model were those developed for Segments 5 and 6 during the summer calibrations. Calibration of the snowmelt phase started with the preparation of inputs on the water content of the snowpack, snowpack density, the snowpack temperature and snowpack dullness. The meaning of dullness is explained in Section 2.1.1 of Appendix B. These initial estimates were made for a time about two weeks prior to the start of snowmelt.

Estimation of the snow water content is described in Appendix D. Snowpack density estimates were based on snowcourse data. The initial snowpack temperature was estimated to be the average ambient temperature for seven days preceeding the start data of the simulation run. The initial dullness was obtained by assuming an average dullness one month prior to the start date of the simulation run, and then manually computing the dullness day by day up to the start date.

The parameters within the snowmelt routine which required calibration were SHADE, ALBEDO and CCFACT. SHADE represents the interception of direct sunlight by trees, and directly affects the shortwave melt, as described in Appendix B. An estimate of SHADE was made based on the percentage of tree cover, keeping in mind the fact that the trees are leafless during spring runoff. It was then adjusted somewhat so that good results were achieved for those situations where shortwave melt appeared to be the major component of the total snowmelt. The final value of SHADE derived was 0.20. With respect to the computation of ALBEDO which is a measure of the reflectance of snow, a different procedure was used for the winter and the spring, as described in Appendix B. The HSPF model is designed to switch from the winter to the spring computational procedure on April 1. This produced difficulties, and as a result the model was modified so that the changeover to the spring albedo formula would occur when nighttime refreezing of the snowpack had ceased.

The convection-condensation factor (CCFACT) directly affects both convection melt and condensation melt. The most suitable value of CCFACT was obtained by trial and error, using particularly data for 1979 when the total melt appeared to result mainly from convection and condensation. The final value derived for CCFACT was 0.75.

The channel routing procedure in the model employs a simple linear reservoir technique. This module was used to route flows along the Boyne River from the hydrometric gauge near Treherne (station 050F010) to the hydrometric gauge near Roseisle (050F011). It was also used to route flows through the Stephenfield Reservoir. The channel routing configuration is shown in Figure E-4. Channel routing procedures are discussed more fully in Appendix F.

### E.4.3 Spring Calibration - SSARR Model

#### E.4.3.1 General Approach

The computed values of the soil moisture index (SMI) at freeze-up, obtained from the summer simulations for 1969, 1975 and 1978, were used as starting basin conditions for the spring calibrations of 1970, 1976 and 1979 respectively.

Calibrations were performed with data averaged over six-hour time periods for the spring runoff of 1970, 1976 and 1979. Data time series representing streamflows, ambient temperatures, dew point temperatures, precipitation amounts, solar radiation totals and winds were used as input for the calibrations.

Under both the single and segmented basin approaches, channel routing of flows was effected from the Boyne River near Treherne to the Boyne River near Roseisle, and through the Stephenfield Reservoir. The routing procedure used was made identical to that used with the HSPF model; this was done deliberately to facilitate a better comparison of the performance of the hydrologic components of these models.

#### E.4.3.2 Calibration Techniques

Calibration of the SSARR model for spring runoff started with the calibration of the snowmelt component of the model. Important parameters of the snowmelt equation determined by calibration were the albedo (A), the convection-condensation melt factor (K), the shortwave radiation melt factor (K′), and the average forest canopy cover (F).

The SSARR snowmelt procedure does not involve modelling of the albedo, i.e. the albedo must be measured or estimated externally and is then input to the model. Also the effect of areal snow cover depletion on snowmelt is not modelled. Since these were considered to be important components affecting snowmelt, an empirical relationship was devised (A. A. Warkentin, pers. comm.) whereby the daily albedo and extent of snow cover was estimated from daily data on temperature and snowfall as follows:

$$A_2 = A_1 - 2(T_a + 4) + 5P$$

where: $A_2$ = albedo on day two (%)
$A_1$ = albedo on day one (%)
$T_a$ = mean ambient temperature on day two (°C)
P = snowfall on day two (mm of water content)

The term $(T_a + 4)$ is set to zero if $T_a$ is less than -4°C. The maximum allowable value of the albedo is 90% and the minimum is 0%. This relationship was tested on the Boyne River data for the spring of 1970, 1976 and 1979.

The constants in the formula were obtained by a trial-and-error process using information on temperature, snowfall and snowcover in the Boyne River watershed during the spring of 1970, 1976 and 1979. Since the albedo computed with this formula will decrease to zero, the areal depletion of snowcover in the latter stages of snowmelt is implicitly included in the formula. It should be noted that this formula was derived for Boyne River snowpacks where water content at the start of melt was between 160 and 180 millimetres. It may not be as applicable to larger or smaller snowpacks or to other watersheds.

Starting values of the albedo were set at 90 percent at the end of the last cold spell of the winter, i.e., at the onset of generally above freezing temperatures. These starting values appeared to be valid for the calibration years when enough fresh snow fell prior to the start date to raise the albedo. The 90 percent starting value would be too high if a long dry spell preceeded the start date. Daily values of the albedo for the snowmelt periods of 1970, 1976 and 1979 were then computed and input to the snowmelt component of the SSARR model. Plots of the computed albedos are shown on Figure E-5.

With the computed daily albedo entered into the snowmelt model, and an estimated percent of leafless tree cover (F) of 10 percent, calibration of the SSARR snowmelt routine was reduced to the determination of appropriate values of the melt factors K and K′. This was done through a trial-and-error process, until the simulated dates for the completion of snowmelt coincided with the estimated dates of the disappearance of the snowpack. The latter dates were estimated from snow cover reports, photos and the time of the peak on small watersheds in the vicinity of the Boyne River. These dates are shown on the left hand side of Table E-3. Snowmelt was simulated separately for the escarpment and for the sub-escarpment using the meteorological inputs for each region described in Appendix D. In the single basin approach, escarpment melt rates were used for the entire watershed since the escarpment area produces by far the most runoff. Under the segmented basin approach, sub-escarpment melt rates were used for Segment 5 and escarpment melts for the other segments.

Calibration of the lands phase of the SSARR model started with the development of appropriate Runoff Percent (ROP) curves. The ROP curves obtained from the summer calibrations were not applicable to the spring runoff simulations because spring runoff was found to be much greater than summer runoff for a given SMI. Development of ROP curves was largely a trial-and-error process, the objective of which was the simulation of the correct volume of spring runoff. For the segmented basin approach, ROP curves were developed for each segment simulated with the SSARR model, i.e. segment 1 & 2, segment 3 & 4, segment 5 and segment 6. For the single basin approach, one ROP curve was developed for use on the entire basin. The spring runoff percent curves for the individual segments and for the entire basin are shown on Figure E-6.

The next step in the calibration was the determination of parameters which affect the timing of runoff, i.e. the Baseflow Percent Curve, the Surface/ Subsurface Distribution Curve and the phase routing (conveyance) parameters. The latter determine the rate of flow through the surface, the subsurface and the groundwater systems. These parameters do not include channel routing of flows.

On the Boyne River, hydrograph analysis shows that the baseflow component of spring runoff is typically small in comparison to the total volume of spring runoff. Furthermore there is a lack of field data to verify the behaviour of groundwater flow. Therefore the baseflow percent curve for spring runoff was estimated. The percent of runoff going to baseflow was estimated to be considerably lower than for summer conditions, due to reduced infiltration caused by frozen ground conditions. The baseflow percent curve used in this study is shown on Figure E-7. This curve was used for individual segments as well as for the entire basin.

The Surface/Subsurface Distribution curves and the phase routing parameters were calibrated together. This was necessary because both have a direct effect on the shape of the simulated hydrograph. These curves and routing parameters were obtained by trial-and-error. The calibrated Surface/Subsurface Distribution curves are shown on Figure E-8. Due to the similarity of the soils and slopes in segments 1 & 2, 3 & 4, and 6, the same Surface/Subsurface curve could be used for all of these segments.

The values of the phase routing parameters for both the segmented and the single basin approaches for the surface, subsurface and baseflow components are shown in Table E-4.

Channel routing of flows along the Boyne River from Treherne to Roseisle and through the Stephenfield Reservoir was done for both the segmented basins approach and for the single basin approach. In order to enable a better comparison of the hydrologic components of the SSARR and HSPF models, the decision was made to use similar routing procedures. The channel routing configuration and the distribution of inflows used for the SSARR model was the same as that used for the HSPF model and is shown on Figure E-4. Channel routing is discussed in more detail in Appendix F.

## E.4.4 Spring Calibration - SLURP Model

### E.4.4.1 General Approach

The SLURP model was calibrated on Roseisle Creek and on the entire Boyne River watershed upstream of Stephenfield.

The model was calibrated on a daily time step using mean daily ambient temperature and daily precipitation as input.

Due to the different nature of summer runoff and spring runoff, parameter values obtained from the summer calibrations were not used.

The procedure used to calibrate Roseisle Creek (Segment 6) was the same as that used to calibrate the entire basin upstream of Stephenfield.

### E.4.4.2 Calibration Techniques

Calibration of the SLURP model on spring runoff involved a trial-and-error procedure to obtain appropriate values for eleven model parameters pertinent to spring runoff. These are asterisked in Table E-5, together with the parameter values obtained from calibrations.

Other parameters inputted to the model included the starting watershed conditions, the externally computed melt factors, and a few other parameters which were estimated rather than calibrated. These are also shown in Table E-5. Variables X4, X6, X20 and X21 were not calibrated since they relate to evapotranspiration data which was not used for the spring simulations. Variable X12 (Table B-1) was not required since a programming change was made to include any effects of rainfall on snowmelt in Variable X11.

The model was found to be very sensitive to variables X8, X10, X11, X13 and X14 to X18. These symbols are explained in Table E-5. The simulated flows were found to be relatively insensitive to the initial storages in the SURFACE and GROUNDWATER reservoirs. The latter likely resulted from oversimplifications in the model which does not properly account for the effects of soil and sub-soil moisture on runoff. The capacity of the SURFACE storage reservoir, variable X9, was set at a high level so that the simulated SURFACE storage would never exceed the capacity of this reservoir. This was necessary because of an over-simplification in the model which assigns all of the excess storage of this reservoir directly to runoff.

The start dates for the simulation runs were April 1 for 1970, March 1 for 1976 and April 1 for 1979. The snowpack values for these dates were input to the model through variable X1 and were the same values used for the HSPF and SSARR model simulations. Any snowfall after these respective dates was input via the time series of daily precipitation data to the model. The starting storages used for the SURFACE and GROUNDWATER reservoirs were those simulated for freeze-up of the previous fall in the summer simulation runs.

The values of the snowmelt factor for each year were externally computed using the relationship shown on Figure B-10. The dew point temperatures used to adjust the melt factor were the prevailing values during the active melt period.

## E.5 DISCUSSION OF RESULTS

### E.5.1 Model Calibration on Summer Data

#### E.5.1.1 HSPF Model

One of the objectives of the flow simulations was to obtain moisture storages at freeze-up, for use as starting conditions for the spring runoff simulations. The October 31 upper zone (UZS), lower zone (LZS) and active groundwater (AGWS) storages obtained for segments 5 and 6 are shown in Table E-6. The computed storages vary considerably from year to year, reflecting the different patterns of precipitation and evapotranspiration for these years.

On Roseisle Creek (Segment 6), the summer simulated runoff volume for 1969 was 10,150 $dam^3$ , considerably lower than the observed volume of 14,050 $dam^3$ . In 1975 the total summer simulated flow was 7100 $dam^3$ , considerably higher than the observed volume of 5000 $dam^3$ . In 1978 the simulated volume of 10,700 $dam^3$ was much higher than the observed volume of 5900 $dam^3$ .

These large errors arose primarily from the difficulties associated with developing representative summer precipitation data for the watershed. Most heavy rainstorms are associated with thunderstorms which produce sharp spatial variations in amount and intensity of rainfall. As discussed in Appendix D, no recorded rainfall intensity data were available within the watershed and the data had to be transposed some distance.

The infiltration rate (INFILT) can be a good indicator on whether there is an error in the precipitation data. Choosing too low an infiltration rate should result in an oversimulated peak and undersimulated baseflow following storm events. If raising the infiltration rate significantly does not result in an underestimate of the peak flow and an overestimate of baseflow, then there is a strong indication of an error in the precipitation estimate. Using this criterion the basin precipitation file was in error for six out of the seven summer storms on Roseisle Creek in 1978.

A second source of error results from the model's procedure for estimating evapotranspiration. The model ignores the complex interaction between the plant stomata and the environment.

In 1969 the model underestimated flows resulting from three major rainstorms which occurred during a 10-day period in late June and early July. Runoff from this multiple storm event was by far the largest that occurred in the summer calibration years. A special effort was made to calibrate the model so as to reproduce this runoff event.

Simulated flows for this event are shown on Figure E-9. The model simulated the first peak well but underestimated the second and third peaks. Results for this event also raised some doubts as to the ability of the model to account for rapid increases in effective drainage area as pothole storages fill.

For Segment 5, fine tuning of the calibration was not possible due to errors in the computed streamflows for this segment. As explained in Section E-2.2, flows for this segment were computed from main stem flows because the segment flows have not been measured. However, it was apparent from the computed segment flows that this segment seldom produces summer runoff, except after major rainstorms. This is logical in view of the sandy soil and the lack of local relief. Calibration of the model on Segment 5, was therefore, limited to a reproducing reasonable seasonal runoff volumes and to keeping the process parameters within the guidelines for this type of watershed.

Although the results of the summer calibrations were not as accurate as had been hoped for, they were still thought to provide a reasonable estimate of starting moisture storages for the spring runoff simulations.

### E.5.1.2 SSARR Model

Table E-7 shows the final computed and estimated values of the Soil Moisture Index at freeze-up for the summer calibration years. These were used as starting values for the spring runoff simulations.

Flow simulations on Roseisle Creek for late June and early July, 1969 are shown on Figure E-9. The simulated flows are not accurate. Some of the errors were likely due to inaccurate watershed precipitation estimates and errors in the calibrated model parameters. However, a significant portion of the error is likely due to the fact that the model does not take rainfall intensity into account. Furthermore, the Runoff Percent/SMI relationship does not explicitly include the increase in the effective drainage area which takes place when depressional storages become filled by rainfall runoff.

Table E-8 contains a summary of the seasonal water balance components for the calibrated segments for each of the calibration years. The average runoff coefficient for the May 1 to October 31 period is only 8.3 percent of the precipitation. The highest seasonal runoff coefficient was 22 percent in Segment 6 during 1969. This was mainly a result of the large rainstorms in late June and early July. As indicated on Table E-8, the error in seasonal generated runoff ranges from -0.7 to +0.6 inches. The simulated actual evapotranspiration averaged 73.8 percent of the input potential evapotranspiration. The last two columns show that the simulated fall Soil Moisture Index values are considerably lower than preceeding values on May 1. This suggests that evapotranspiration and runoff combined generally exceed the moisture input during the summer.

#### E.5.1.3 SLURP Model

Components of the SLURP model water balance for the summer calibrations are shown in Table E-9. A good agreement was achieved between simulated and observed runoff volumes. However, the computed moisture storages at freezeup appear to be abnormally low. An attempt was made to rectify this by reducing evapotranspiration (variable X20), but this resulted in an overestimate of runoff. To compensate for this, the runoff factors (X8 and X16) were reduced. This, in turn, led to a gross underestimation of storm runoff. Other solutions were attempted but no adequate ones were found. The problem appeared to result from basic inadequacies in the model.

The calibration parameters obtained (Table E-2) were considerably different for 1969 than for 1975 and 1978. This was due to the influence of the major rainstorms which occurred in late June and early July of 1969, which resulted in unusual runoff characteristics for this year. It was not possible to achieve good results for all three years using one set of calibration parameters. This points to a shortcoming of the model, namely its inability to account for the effects of varying rainfall intensities and its inability to account for non-linear relationships between basin moisture storages, infiltration and runoff.

Results of SLURP model calibrations on Roseisle Creek (Segment 6) were very similar to those obtained for the entire basin upstream of Stephenfield, except that model parameters were somewhat different because of the generally higher runoff coefficients experienced in this sub-basin. Calibration difficulties encountered were very similar to those for the entire watershed.

### E.5.2 Model Calibration on Spring Data

#### E.5.2.1 General Comments

The simulated and recorded flows for the spring calibrations of 1970, 1976 and 1979 are shown on Figures E-10 to E-12 for Roseisle Creek (Segment 6) and on Figures E-13 to E-15 for the Boyne River near Stephenfield. The simulated flows shown on Figures E-13 to E-15 are based on the single basin approach for all models.

All the models tested produced large differences between simulated and recorded flows for 1976. Investigations revealed that this was likely due to abnormal winter evaporation-sublimation losses from the snowpack caused by frequent light thawing and refreezing, particularly in February. Because the 1976 conditions were considered to be very unusual, and because the spring runoff was relatively low, no further effort was made to obtain a good calibration for that year. Better results for 1976 might have been obtained by running a snow accumulation and ablation subroutine, but this was not attempted. Emphasis was, instead, placed on calibrating the models for the type of winter conditions which occur most frequently in Southern Manitoba and which are most conducive to spring flooding, i.e. winters with consistently below freezing temperatures and low sublimation losses from the snowpack. The models were therefore largely calibrated on data for the 1970 and 1979 floods.

The calibrated flows for all models were generally in close agreement with the recorded flows for the spring of 1970. However, it was not possible to calibrate the models to achieve good results for 1979, particularly with respect to the peak flow near Stephenfield. Changing model parameters to improve simulated flows for 1979 generally resulted in poorer results for 1970. It was surmised that the tall, slender shape of the 1979 recorded flow hydrograph near the peak was somewhat unusual, possibly due to the build-up and release of water behind ice or snow dams. The decision was therefore made to retain the model parameters which produced accurate simulated flows for 1970 and which led to an underestimate of the peak flow in 1979.

The calibration results on Roseisle Creek were quite similar in most respects as those for the entire basin. Runoff tended to start somewhat early, particularly where the energy budget was used for snowmelt. As in the entire basin, simulated runoff on Roseisle Creek was too high for 1976, somewhat too low for 1979, and quite accurate for 1970. This pattern contrasts with the currently used MANAPI model which greatly overestimated 1979 flows on Roseisle Creek but was fairly accurate for the entire watershed. Simulated flows for Roseisle Creek were only slightly more accurate than those for the entire watershed near Stephenfield, although this varied somewhat from model to model. This would suggest that the channel routing required for the full basin did not seriously affect the accuracy of the calibrations.

A comparison of model calibration results using the single basin approach, based on deviations between simulated and recorded flows for the calibration years of 1970 and 1979, indicated that the MANAPI model gave the best results, followed by the HSPF, SSARR and SLURP models.

The accuracy of calibrated flows for 1970 and 1979 using the segmented basin approach with the HSPF and SSARR models was no better than that using the single basin approach. This may have been due to inaccuracies associated with the extrapolation of model process parameters from Segment 5 and 6 (HSPF model) and Segments 1 & 2 and 6 (SSARR model) to the other segments.

Factors contributing to inexact model calibrations included the following:

i) Imprecise meteorological data inputs

ii) Imprecise streamflow data, including the possibility of outflows from the Boyne River Watershed near Rathwell under high flow conditions

iii) Inaccuracies in the hydrologic models, including physical processes not modelled or poorly modelled

iv) Inaccuracies in the streamflow routing models

v) Incomplete information on the physiographic characteristics of the watershed

vi) Imperfect skills of the modellers and lack of sufficient time

#### E.5.2.2 Snowmelt Calibration

With respect to the HSPF and SSARR models, deficiencies of the snowmelt equations may have contributed significantly to calibration difficulties. These melt equations were developed for deep mountain snowpacks; their applicability to shallow prairie snowpacks has not been thoroughly tested. The albedo calculation procedures are not well tested. The negative heat storage of the snowpack is neglected in the SSARR model and poorly computed in HSPF. The effects of cloud cover on long-wave radiation losses is only included in HSPF. The reduction in rates of convection and condensation melt due to atmospheric inversions near the snow surface is not explicitly accounted for.

Calibration of the snowmelt equations of the HSPF and SSARR models was difficult due to the lack of data on the actual melt occurring during each simulation time step. This made it necessary to determine the coefficients in the equations from estimates of the date of completion of snowmelt and from streamflow data. The probable errors in the simulated snowmelt likely contributed to the difficulty in calibrating the lands phase of the models.

Due to the difficulties of calibrating the snowmelt equations of the HSPF and SSARR models the question arose as to whether the relatively high cost of calibrating these equations was justified. It is noted that these snowmelt calibrations and the subsequent use of these equations require the relatively costly preparation of hourly and hence 6-hourly meteorological data files for at least four weather elements. To help answer this question, a comparison was made between snowmelt computed with the snowmelt equations and snowmelt computed using a simple melt factor. The melt factor technique involved computing daily degree-days above a base temperature and specifying the melt factor which is the amount of melt per degree-day. Various trials were run to determine the most appropriate base temperature and melt factor. Using this simple procedure and the SSARR snowmelt equations, dates of snowmelt completion were then computed for five different years and were compared to the estimated actual dates of snowmelt completion. The results are shown on Table E-10. The results show that the simple melt factor approach compares quite favourably with the SSARR energy budget for most years, at least with respect to cumulative seasonal melt. However, the melt factor approach performed very poorly for 1979 when unusually high dew point temperatures occurred during the snowmelt period. The four day error in the date of snowmelt completion is very significant for a large snowpack which melted in three to four days. This analysis indicates that the degree-day melt factor approach to snowmelt computation may work well in most instances, but is not always reliable. Therefore, the most accurate approach is to use the energy budget equations.

### E.5.2.3 Lands Phase Calibration

The simulated flows for 1970 and 1979 shown on Figures E-10, E-12, E-13 and E-15 show that the models were reasonably well calibrated.

A few important inadequacies in the lands phase of the models compounded the difficulties of achieving a good model calibration. None of the models tested have the capability of simulating ground frost and its effects on soil moisture, infiltration and runoff. This continues to be a difficult area to model because of a lack of basic knowledge of the physical processes involved. Another basic inadequacy of the models is their inability to simulate changes in the effective drainage area of a watershed which result from significant changes in the moisture conditions of the watershed. This was particularly a problem with the HSPF and SLURP models. With the SSARR model the effect of a changing drainage area could be accounted for implicitly by adjusting the Runoff Percent vs SMI relationship. However, this was difficult to do since there are other factors which affect the shape of this curve. With the currently used MANAPI model, it may be that this effect is implicitly accounted for through the choice of optimum exponents on the Total Precipitation and API variables in the runoff equation.

The lack of field data on overland flow, interflow and groundwater flow made it necessary to estimate these flow components from hydrograph analysis and soil/subsoil characteristics. For the HSPF model it was necessary to study summer runoff events in order to obtain model process parameters for the lands phase of the model. There was not enough information available from the spring calibration data to properly estimate these parameters. Use of the summer-based process parameters for the spring calibrations may not be entirely appropriate and may have affected the calibration results of the HSPF model.

### E.5.2.4 Conveyance

With respect to the time distribution of overland flow, only the HSPF model is suited to the input of physiographic basin features directly related to this phenomenon.

The slope and the length of the overland flow plane were readily available from topographic maps and did not appear to pose a limitation on model calibration. Data on land use and vegetation required to estimate the surface roughness was also readily available and did not pose a limitation. Other conveyance parameters for the HSPF model and all the conveyance parameters for other models had to be estimated and then adjusted through the calibration process.

The lack of channel routing on Roseisle Creek may have created some calibration difficulties. Despite the fact that this Creek has a relatively small drainage area, it has two distinctly separate topographic features. The upstream portion is a relatively flat plateau while the downstream portion, where the stream flows off the escarpment, is quite steeply sloped. The stream sometimes produces a second peak for a given moisture input, due to the slower runoff from the flatter upstream area. It was not possible to simulate this effect with a single watershed approach and no channel routing.

### E.5.3 Observation

The thoroughness of the model calibration was limited by the time and cost constraints of the Agreement Respecting Flood Forecasting, and by the input data and model limitations described earlier. Nevertheless, the calibrations are considered to be fairly good considering the present state-of-the-art of river flow forecasting. The calibration of models was considered to be rigorous enough to enable a valid comparison of relative model performance on model verification data.

## E.6 LITERATURE CITED

1. Crawford, N.H. and Linsley, R.K., 1966. Digital Simulation in Hydrology - Stanford Watershed Model IV, T.R. 39. Department of Civil Engineering, Stanford University, California.
2. Donigian, A.S. Jr. and Davis, H.H. Jr., 1978. User's Manual for Agricultural Runoff Management (ARM) Model. U.S. Environmental Protection Agency, Athens, Georgia 30605.
3. Johanson, R.C., Imhoff, J.C., Davis, H.H. Jr., Kittle, J.L. Jr., Donigian, A.S. Jr., 1981. User's Manual for Hydrological Simulation Program - Fortran (HSPF), Release 7.0. U.S. Environmental Protection Agency, Athens, Georgia, 30613.

## Table E-1
## Calibration Parameters for Summer Runoff
## HSPF Model

| Parameter Variable Name and Description (Stanford Name) | | Type of Parameter | Estimation Technique | VALUES OBTAINED | |
|---|---|---|---|---|---|
| | | | | Segment 5 | Segment 6 |
| FOREST | portion of watershed that transpires in winter | process | default | 0.0 | 0.0 |
| LZSN | lower zone nominal storage | process | calibrated | 10.0 | 6.0 |
| INFILT | infiltration rate | physical | calibrated | 0.60 | 0.05 |
| LSUR(L) | length of overland flow plane | physical | estimated using a grid drawn on a topographic map | 2400 | 2400 |
| SLSUR(SS) | average overland flow slope | physical | estimated using a grid | .0055 | .0550 |
| KVARY(KV) | nonexponential decay of groundwater recession | process | default | 0.0 | 0.0 |
| AGWRC(KK24) | basic groundwater recession rate | physical | logarithmic calculation during baseflow | 0.900 | 0.900 |
| PETMAX | air temperature at which ET will be reduced if snow is being considered | process | default | 40.0 | 40.0 |
| PETMIN | | process | default | 35.0 | 35.0 |
| INFEXP | exponent in the infiltration equation | process | default | 2.0 | 2.0 |
| INFILD | ratio between maximum and minimum infiltration capacities over the PLS | process | default | 1.5 | 1.5 |
| DEEPER (K24L) | fraction of groundwater flow which will enter inactive storage | process | default | 0.0 | 0.0 |
| BASETP (K24EL) | baseflow ET | process | calibrated | 0.0 | 0.0 |
| AGWETP | groundwater ET | process | calibrated | 0.0 | 0.0 |
| CEPSC (EPXM) | interception storage capacity | process | calibrated | 0.05 | 0.05 |
| UZSN | upper zone nominal storage | process | calibrated | 0.06 | 0.30 |
| NSUR(NN) | Manning's N for overland | physical | estimated from roughness table | 0.40 | 0.40 |
| INTFW (INTER) | interflow inflow parameter | process | calibrated | 2.0 | 2.0 |
| IRC(IRC) | interflow recession constant | physical | hydrograph subtraction | 0.50 | 0.50 |
| LZETP(KB) | lower zone ET | process | calibrated | monthly | monthly |

### Table E-2
### Calibration Parameters for Summer Runoff
### SLURP Model

| Variable Name | Description | Values Obtained 1969 | 1975 | 1978 |
|---|---|---|---|---|
| X6 | Proportion of Evapotranspiration from the SURFACE reservoir | 0.7 | 0.7 | 0.7 |
| X8 | Proportion of SURFACE Reservoir going to Runoff | .015 | .003 | .004 |
| X10 | Proportion of SURFACE Reservoir going to Percolation | .04 | .07 | .07 |
| X16 | Proportion of GROUNDWATER Reservoir going to Runoff | .004 | .004 | .004 |
| X20 | Multiplying Factor for Input Potential Evapotranspiration | 1.0 | 1.0 | 1.0 |

## Table E-3
## Dates of Snowpack Disappearance for Calibration Years

| Year | Observed (Estimated) | Modelled - SSARR Model Escarpment | Modelled - SSARR Model Sub-escarpment | Remarks on Type of Snowmelt |
|---|---|---|---|---|
| 1970 | April 27 | April 29 | April 25 | Mainly radiation for the first few days. Predominantly convection for the remainder. |
| 1976 | April 6 | April 5 | April 4 | Mainly radiation. Some convection. |
| 1979 | April 20 | April 21<br>1800 hours | April 21<br>1200 hours | Predominantly convection and condensation. |

### Table E-4
### Surface, Subsurface and Baseflow Routing Parameters Resulting from Spring Runoff Calibrations SSARR Hydrologic Model Boyne River Watershed

| Segment | Surface | | Subsurface | | Baseflow | |
|---|---|---|---|---|---|---|
| | Phases (n) | Hours/phase (Ts) | Phases (n) | Hours/Phase (Ts) | Phases (n) | Hours/Phase (Ts) |
| 1& 2 | 3 | 8 | 3 | 40 | 2 | 70 |
| 3 & 4 | 4 | 20 | 3 | 50 | 2 | 70 |
| 5 | 4 | 10 | 4 | 40 | 2 | 70 |
| 6 | 4 | 10 | 3 | 50 | 2 | 70 |
| Entire Basin at Stephenfield (Single Basin) | 4 | 20 | 3 | 50 | 2 | 80 |

## Table E-5
## Calibration Parameters for Spring Runoff
## SLURP Model

| Variable Number | Description | Values Obtained | | |
|---|---|---|---|---|
| | | 1970 | 1976 | 1979 |
| X1 | Initial Water Content of the Snowpack (MM) | 165 | 115 | 175 |
| X2 | Initial Water Content of the SURFACE Reservoir (MM) | 7 | 4 | 0 |
| X3 | Initial Water Content of the GROUNDWATER Reservoir (MM) | 6 | 14 | 22 |
| X5 | Rainfall Adjustment Factor | 1 | 1 | 1 |
| X7* | Rain/Snow Division Temperature (°C) | 0.8 | 0.8 | 0.8 |
| X8* | Proportion of SURFACE Storage Reservoir Going to Runoff | .07 | .07 | .07 |
| X9 | Capacity of SURFACE Storage Reservoir (MM) | 150 | 150 | 150 |
| X10* | Proportion of surface Storage Reservoir going to Percolation | .15 | .15 | .15 |
| X11* | Snowmelt Factor in Millimetres of Melt per Degree Celsius above the Snowmelt Threshold Temperature | 8.0 | 6.5 | 15.5 |
| X13* | Snowmelt Threshold Temperature (°C) | 0.8 | 0.8 | 0.8 |
| X14* | Proportion of SNOWMELT Reservoir going to Runoff | .06 | .06 | .06 |
| X15* | Proportion of SNOWMELT Reservoir going to Percolation | .10 | .10 | .10 |
| X16* | Proportion of GROUNDWATER Reservoir going to Runoff | .005 | .005 | .005 |
| X17* | Lag Time for Runoff from SURFACE Reservoir (Days) | 1 | 1 | 1 |
| X18* | Lag Time for Runoff from SNOWMELT Reservoir (Days) | 1 | 1 | 1 |
| X19* | Lag Time for Runoff from GROUNDWATER Reservoir (Days) | 4 | 4 | 4 |

* - Parameter adjusted during calibration

### Table E-6
### Moisture Storages at Fall Freeze-Up
### HSPF Model
### (Data in Millimetres)

| Year | Segment | Upper Zone Storage | Lower Zone Storage | Active Ground-Water Storage |
|---|---|---|---|---|
| 1969 | 5 | 0.05 | 100.6 | 0.00 |
| | 6 | 0.41 | 86.2 | 0.00 |
| 1975 | 5 | 0.08 | 95.8 | 0.00 |
| | 6 | 0.25 | 99.0 | 0.00 |
| 1978 | 5 | 0.13 | 127.2 | 0.00 |
| | 6 | 0.25 | 90.5 | 0.00 |

### Table E-7
### Values of Soil Moisture Index at Freeze-up
### SSARR Model

| Segment | 1969 | 1975 | 1978 |
|---|---|---|---|
| 1 & 2 | 3.0 | 3.7 | 2.7 |
| 3 & 4 (Estimated) | 3.1 | 3.7 | 3.3 |
| 5 (Estimated) | 3.0 | 3.6 | 3.3 |
| 6 | 2.7 | 3.4 | 2.8 |
| Entire Watershed | 3.1 | 3.6 | 3.3 |

### Table E-8
### Water Balance Components for SSARR Model Summer Calibrations
### (May 1 to October 31)

| Year | Basin | Total Precipitation (mm) | Recorded Runoff (mm) | Simulated Minus Recorded Runoff (mm) | Error in Runoff as a Percentage of Precipitation (%) | Input Potential Evapotrans-piration (mm) | Simulated Actual Evapotrans-piration (mm) | May 1st SMI (mm) | Oct. 31st SMI (mm) |
|---|---|---|---|---|---|---|---|---|---|
| 1969 | Entire Basin | 333 | 33 | -8 | 2 | 467 | 343 | 114 | 79 |
| 1969 | Segment 1 & 2 | 345 | 31 | -9 | 3 | 467 | 361 | 114 | 76 |
| 1969 | Segment 6 | 333 | 74 | -18 | 5 | 467 | 322 | 114 | 68 |
| 1975 | Entire Basin | 394 | 16 | +5 | 1 | 574 | 432 | 152 | 91 |
| 1975 | Segment 1 & 2 | 422 | 10 | +15 | 4 | 574 | 455 | 152 | 94 |
| 975 | Segment 6 | 368 | 30 | +13 | 3 | 574 | 391 | 152 | 86 |
| 1978 | Entire Basin | 427 | 23 | -1 | 0.3 | 574 | 437 | 114 | 84 |
| 1978 | Segment 1 & 2 | 416 | * | * | * | 574 | - | 114 | 68 |
| 1978 | Segment 6 | 401 | 35 | +10 | 3 | 574 | 401 | 114 | 71 |
|  | MEANS | 382 | 32 | 10 (Absolute) | 2.7 | 538 | 393 | 127 | 80 |

* - Recorded flows not available from July to October

## Table E-9
## Water Balance Components for SLURP Model Summer Calibrations Boyne River Watershed Upstream of Stephenfield (May 1 to October 31)

**Data in Millimetres**

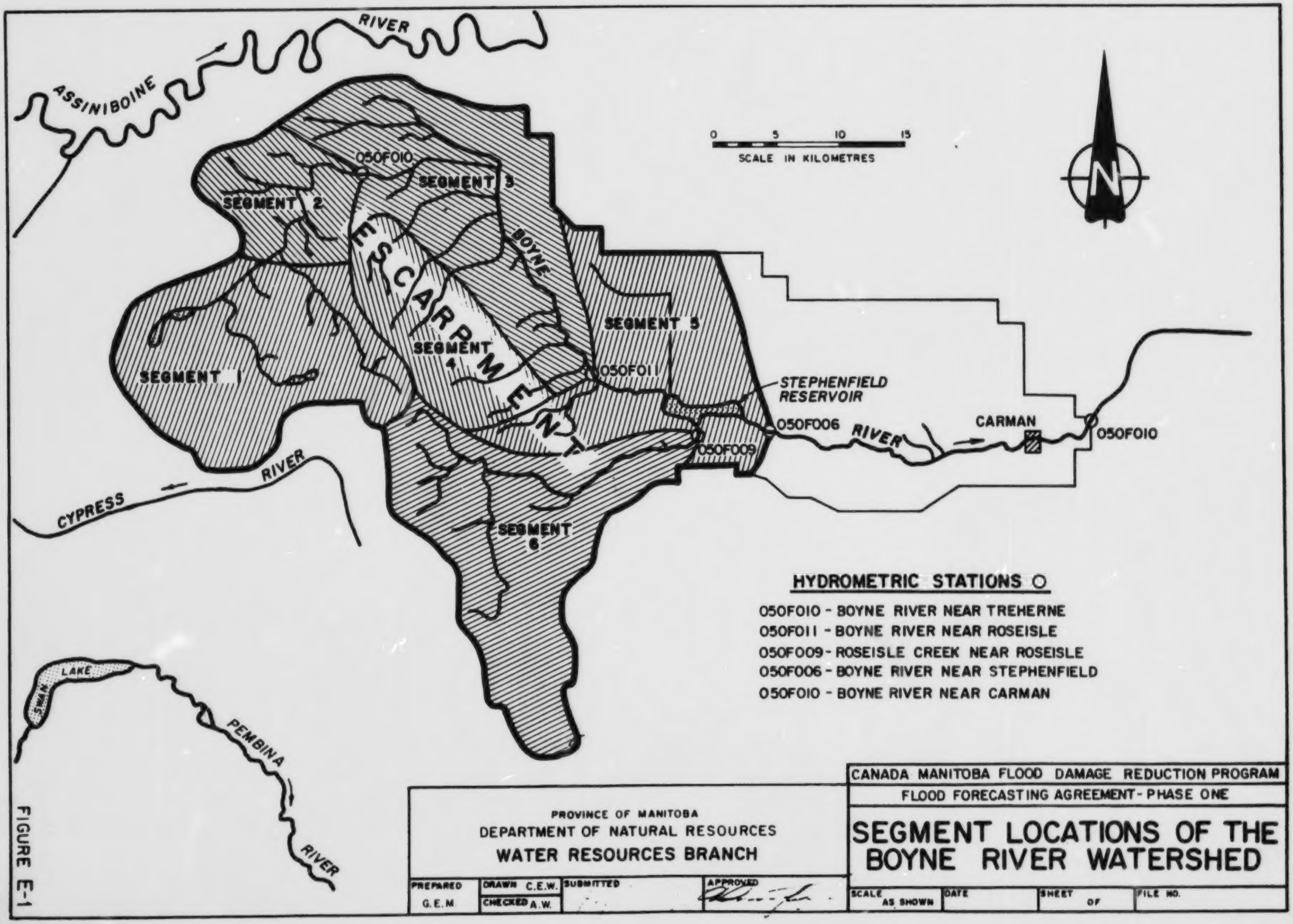
| Year | Total Precipitation | Recorded Runoff | Simulated Runoff | Input Potential Evapotranspiration | Simulated Actual Evapotranspiration | May 1st Storages | | October 31st Storages | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Surface | Groundwater | Surface | Groundwater |
| 1969 | 332 | 43 | 45 | 467 | 373 | 30 | 50 | 7 | 6 |
| 1975 | 393 | 17 | 18 | 574 | 445 | 40 | 50 | 4 | 14 |
| 1978 | 427 | 22 | 23 | 574 | 451 | 20 | 50 | 0 | 22 |

### Table E-10
### Comparison of Dates of Snowmelt Completion
### SSARR Model Versus Simple Melt Factor Approach

| Year | Snowpack (mm of water) | Date of Snowpack Completion | | |
|---|---|---|---|---|
| | | Observed | *SSARR Model | **Melt Factor Approach |
| 1969 | 125 | April 10 | April 10 | April 11 |
| 1970 | 165 | April 27 | April 29 | April 27 |
| 1971 | 142 | April 10 | April 10 | April 10 |
| 1974 | 149 | April 19 | April 17 | April 19 |
| 1979 | 175 | April 20 | April 21 | April 24 |

* - Energy budget equations were used.

** - Melt factor used was 1.41 mm/degree-day Celsius.
Base temperature used was -1.1 degrees Celsius.


ASSINIBOINE
RIVER
CYPRESS
RIVER
SWAN
LAKE
PEMBINA
RIVER
0 5 10 15
SCALE IN KILOMETRES
N
SEGMENT 1
SEGMENT 2
SEGMENT 3
SEGMENT 4
SEGMENT 5
SEGMENT 6
ESCARPMENT
BOYNE
RIVER
050F010
050F011
050F009
050F006
STEPHENFIELD RESERVOIR
CARMAN
050F010
HYDROMETRIC STATIONS ○
050F010 - BOYNE RIVER NEAR TREHERNE
050F011 - BOYNE RIVER NEAR ROSEISLE
050F009 - ROSEISLE CREEK NEAR ROSEISLE
050F006 - BOYNE RIVER NEAR STEPHENFIELD
050F010 - BOYNE RIVER NEAR CARMAN

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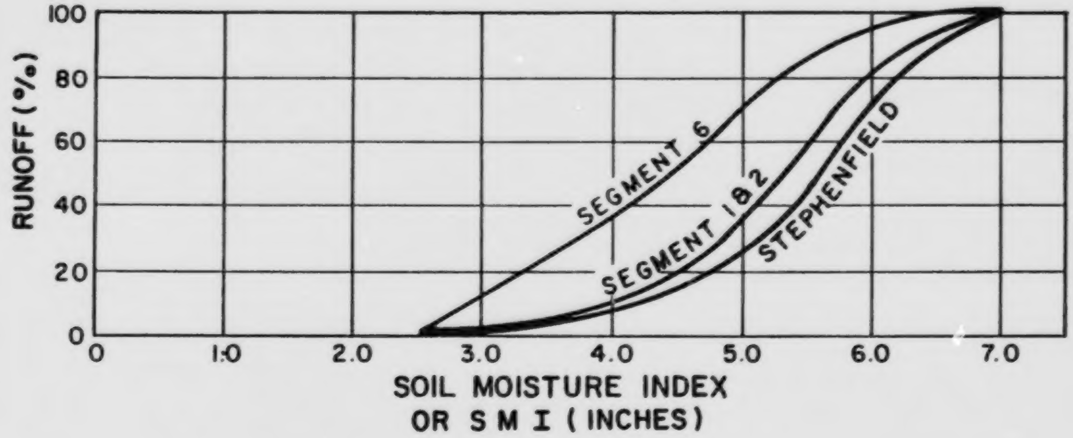
SEGMENT LOCATIONS OF THE BOYNE RIVER WATERSHED

| PREPARED | DRAWN C.E.W. | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| G.E.M. | CHECKED A.W. | | | AS SHOWN | | OF | |

FIGURE E-1

## RUNOFF PERCENT CURVE



## SOIL DRYING CURVE

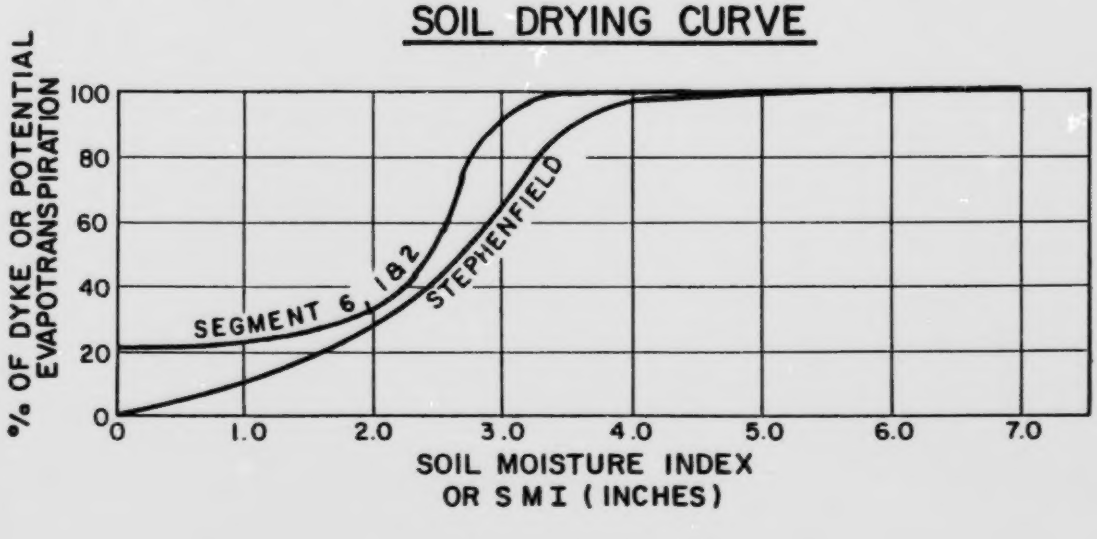


FIGURE E-2

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### RUNOFF PERCENT & SOIL DRYING CURVES USED FOR SUMMER SIMULATIONS - SSARR MODEL

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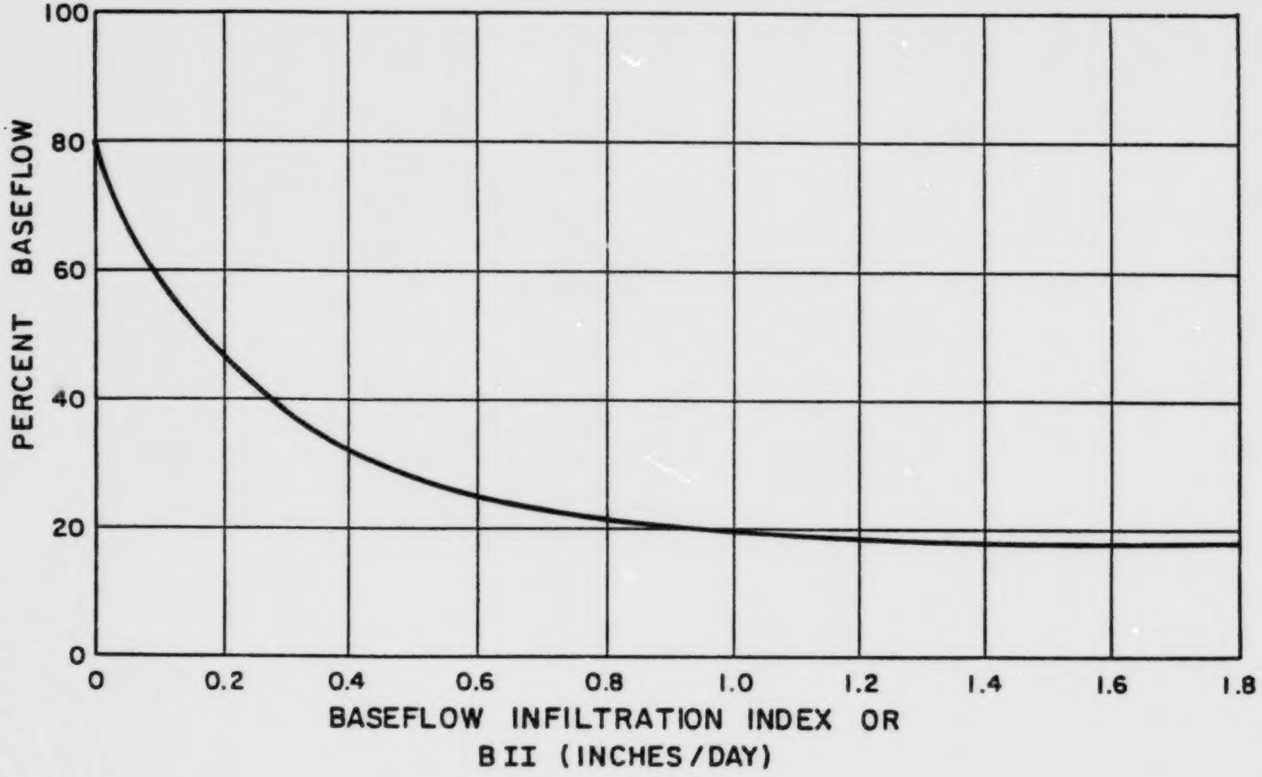
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED D.E.K. | DRAWN<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE | SHEET OF | FILE NO. |



**NOTE:**
DIRECT RUNOFF = (TOTAL RUNOFF) X (1-PERCENT BASEFLOW/100)

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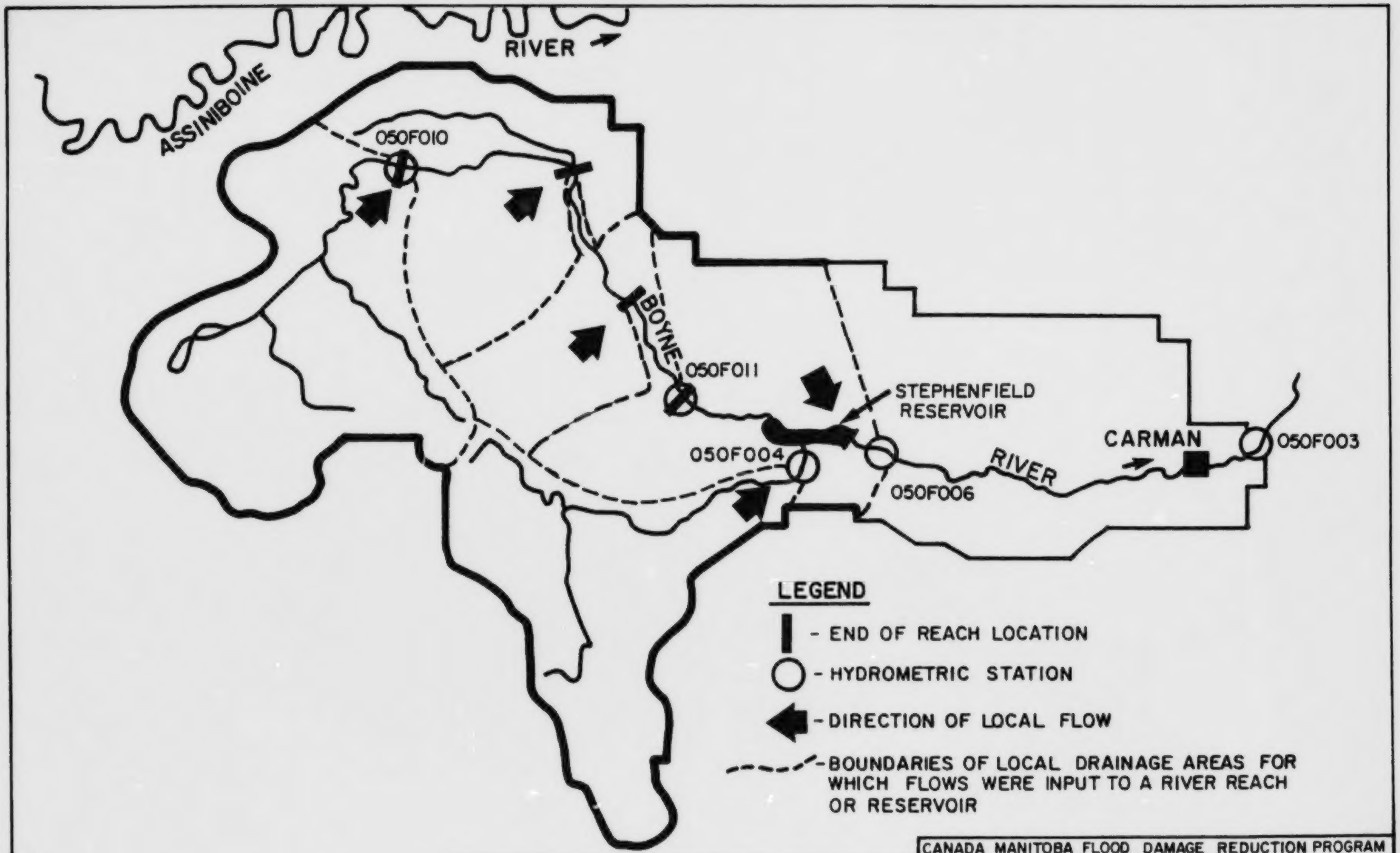
## BASEFLOW PERCENT CURVE FOR SUMMER CALIBRATIONS SSARR MODEL

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| D.E.K. | CHECKED A.W. | | | AS SHOWN | | OF | |

FIGURE E-3


ASSINIBOINE
RIVER
05OF010
BOYNE
05OF011
STEPHENFIELD RESERVOIR
05OF004
05OF006
RIVER
CARMAN
05OF003
LEGEND
- END OF REACH LOCATION
- HYDROMETRIC STATION
- DIRECTION OF LOCAL FLOW
- BOUNDARIES OF LOCAL DRAINAGE AREAS FOR WHICH FLOWS WERE INPUT TO A RIVER REACH OR RESERVOIR

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## CHANNEL ROUTING CONFIGURATION FOR SPRING CALIBRATION - HSPF MODEL

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| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| G.E.M. | CHECKED A.W. | | | AS SHOWN | | OF | |

FIGURE E-4

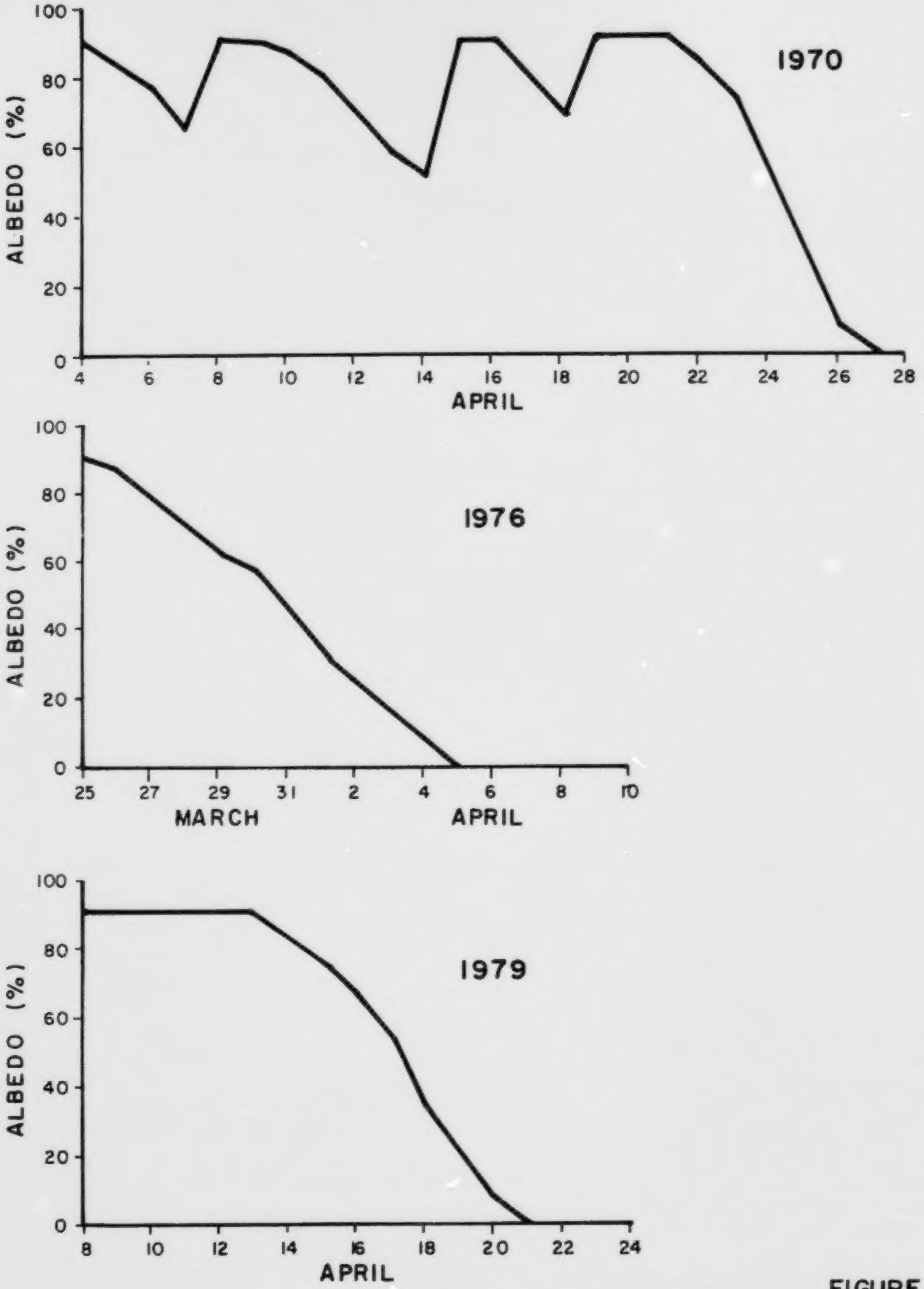

FIGURE E-5

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## DAILY ALBEDOS COMPUTED FOR THE SSARR SNOWMELT MODEL

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED D.E.K. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 85 05 13 | SHEET OF | FILE NO. |

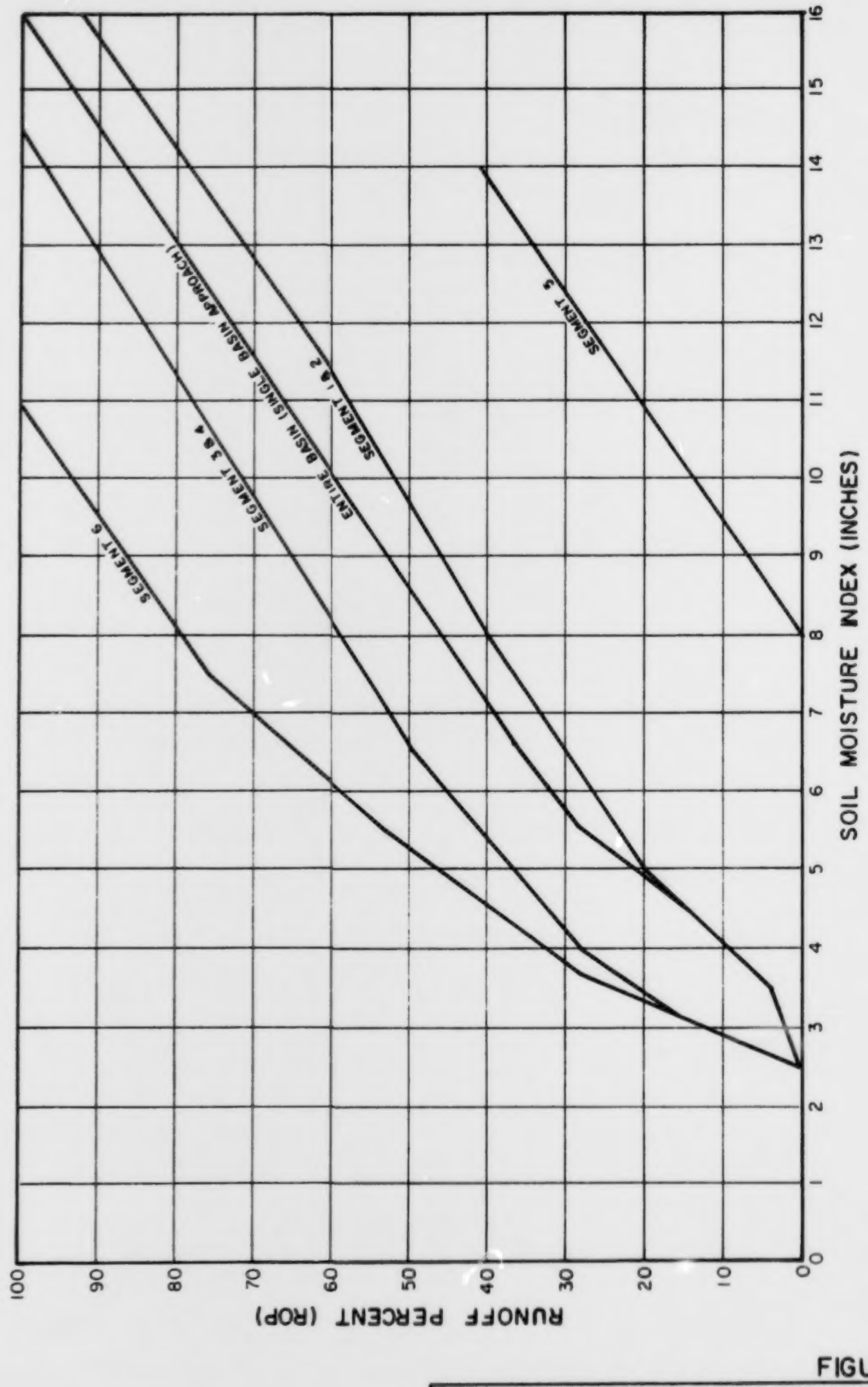

FIGURE E-6

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## SPRING RUNOFF PERCENT CURVES
## SSARR HYDROLOGIC MODEL

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| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED D. E. K. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 85 05 13 | SHEET OF | FILE NO. |

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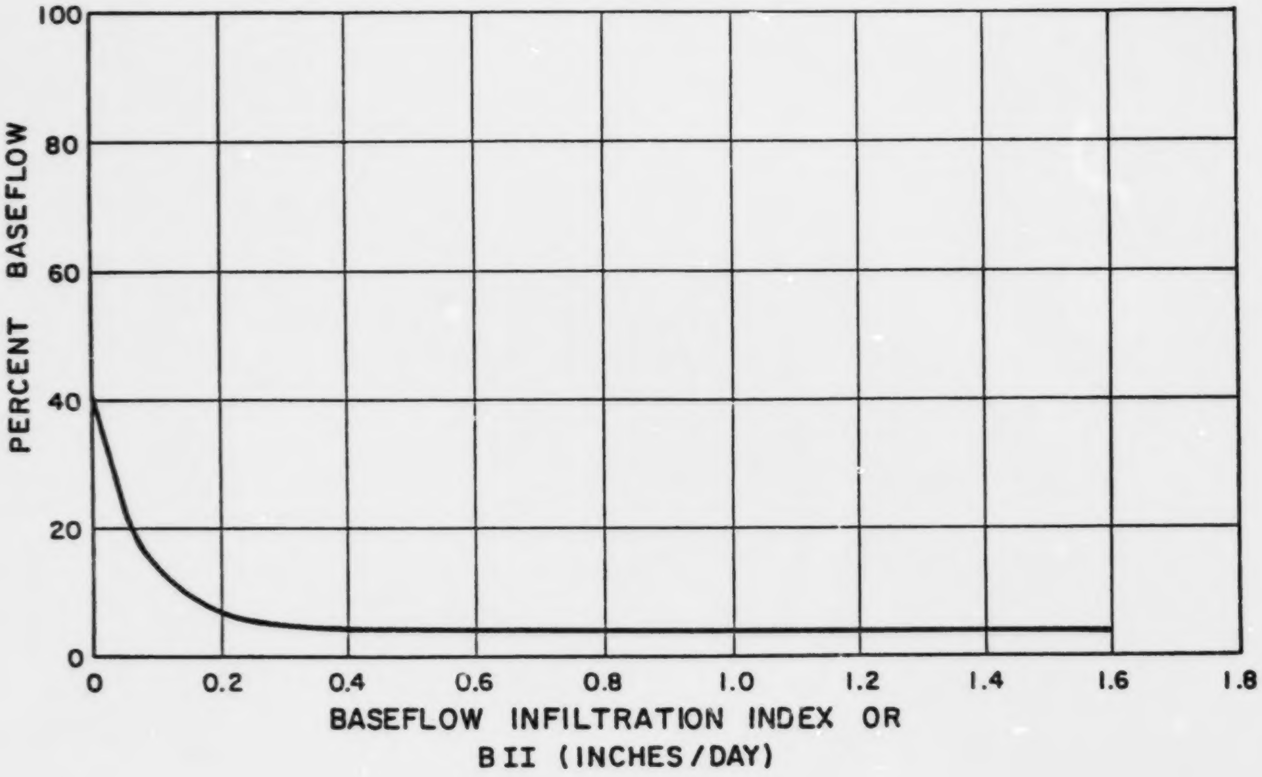

**NOTE:**

DIRECT RUNOFF = (TOTAL RUNOFF) X (1-PERCENT BASEFLOW/100)

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# BASEFLOW PERCENT CURVE FOR SPRING CALIBRATIONS SSARR MODEL

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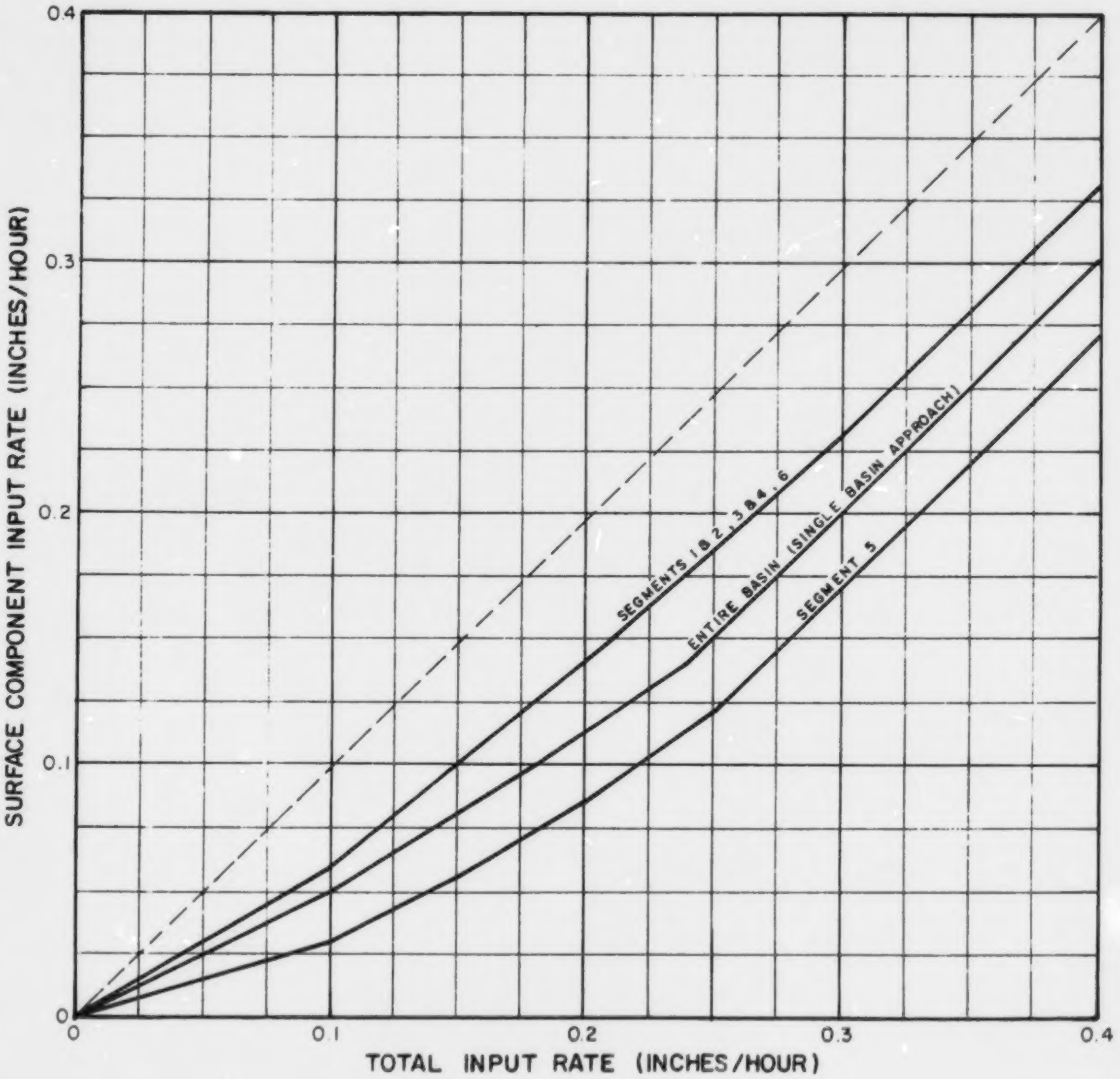
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PREPARED<br>D.E.K. | DRAWN<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE | SHEET OF | FILE NO. |

FIGURE E-7



**BOYNE RIVER WATERSHED**

FIGURE E-8

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

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## SURFACE / SUBSURFACE DISTRIBUTION CURVES BASED ON SPRING CALIBRATIONS SSARR MODEL

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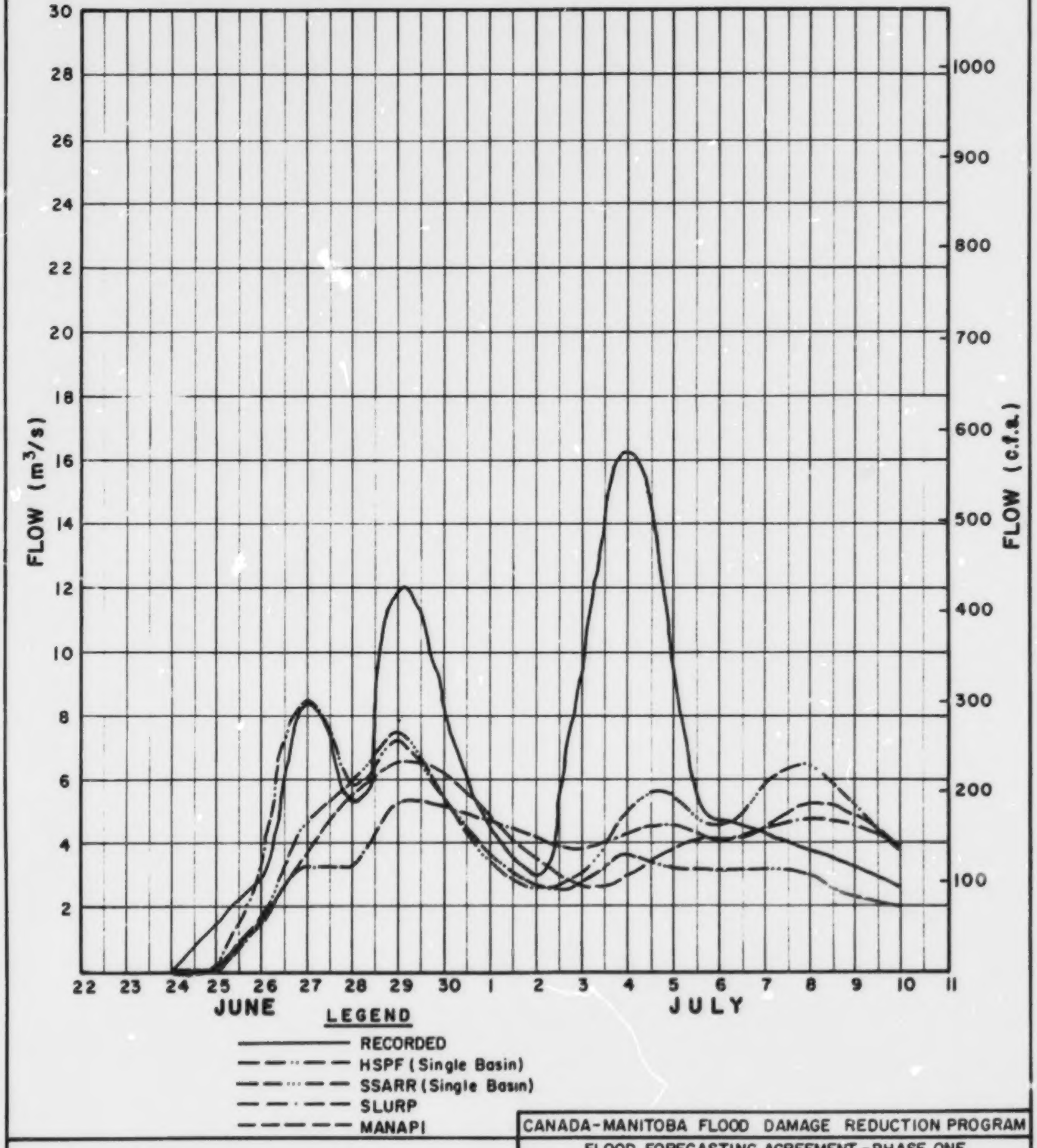

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

## SIMULATED AND RECORDED FLOWS ROSEISLE CREEK NEAR ROSEISLE

CALIBRATION - JUNE, JULY - 1969

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

PREPARED E.L.B.
DRAWN M.M
CHECKED A.W.
SUBMITTED
APPROVED
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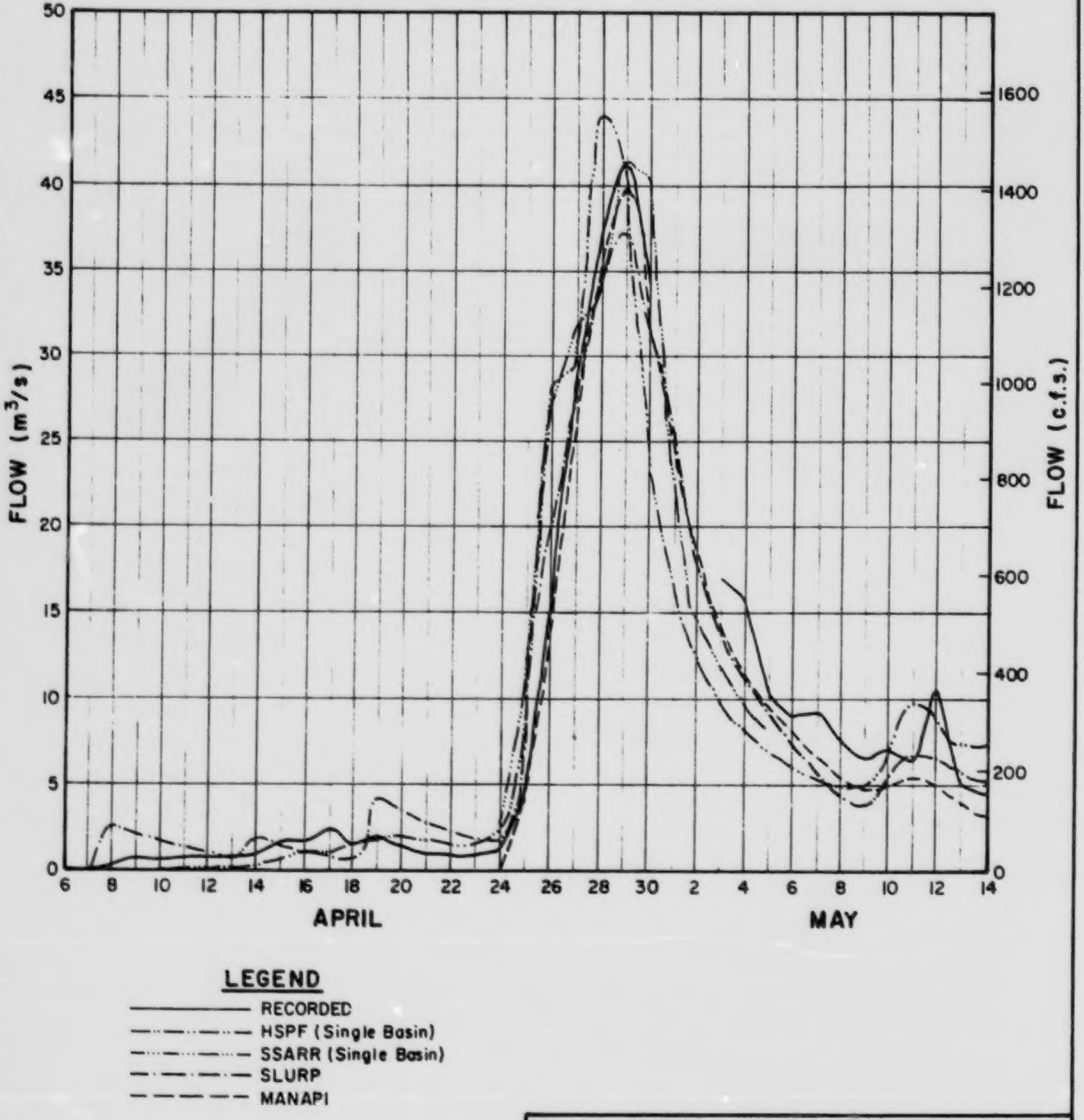
FIGURE E-9



CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
**WATER RESOURCES BRANCH**

## SIMULATED AND RECORDED FLOWS
## ROSEISLE CREEK NEAR ROSEISLE
## CALIBRATION – SPRING OF 1970

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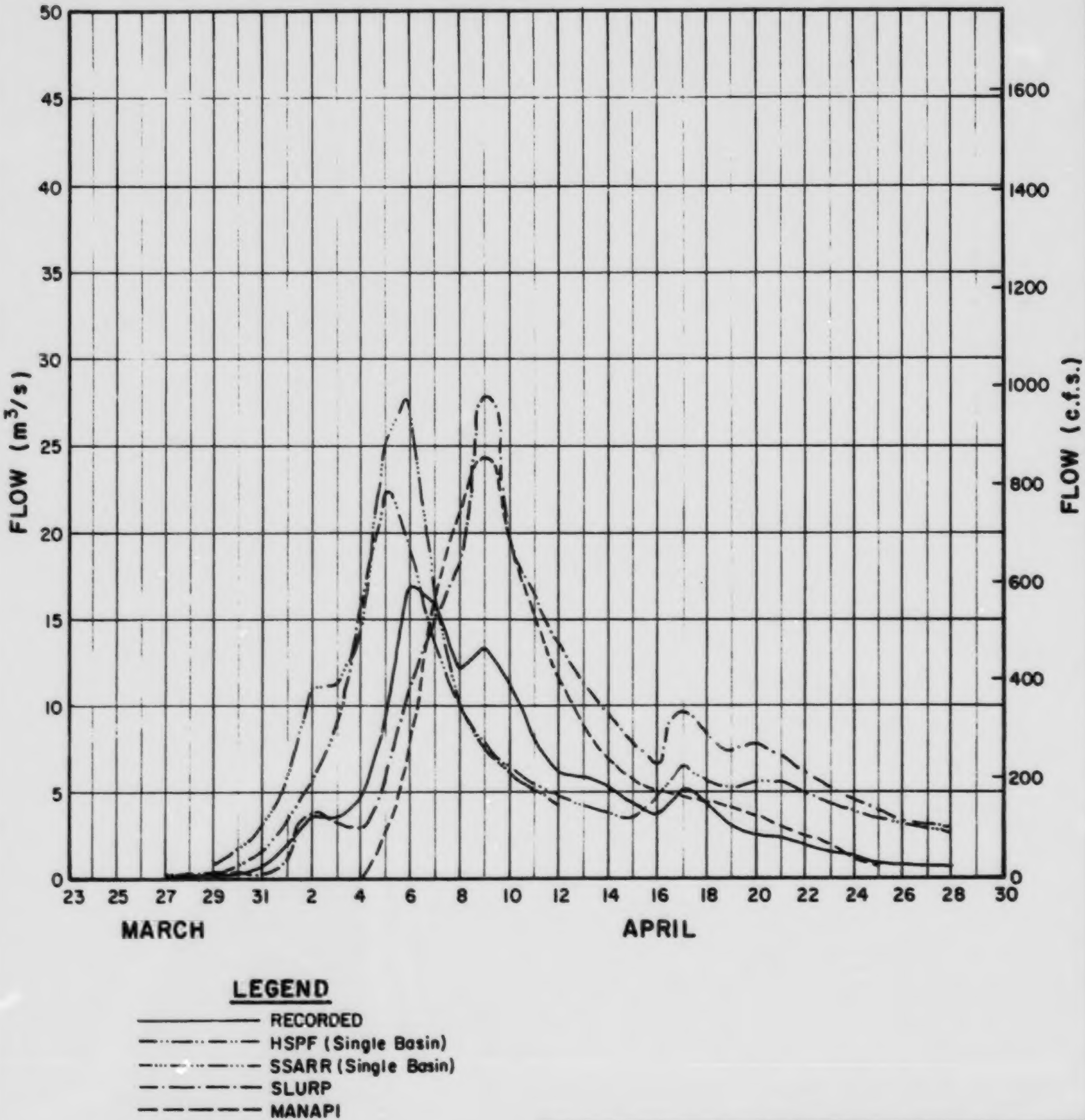
| PREPARED E.L.B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

FIGURE E-10



CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA

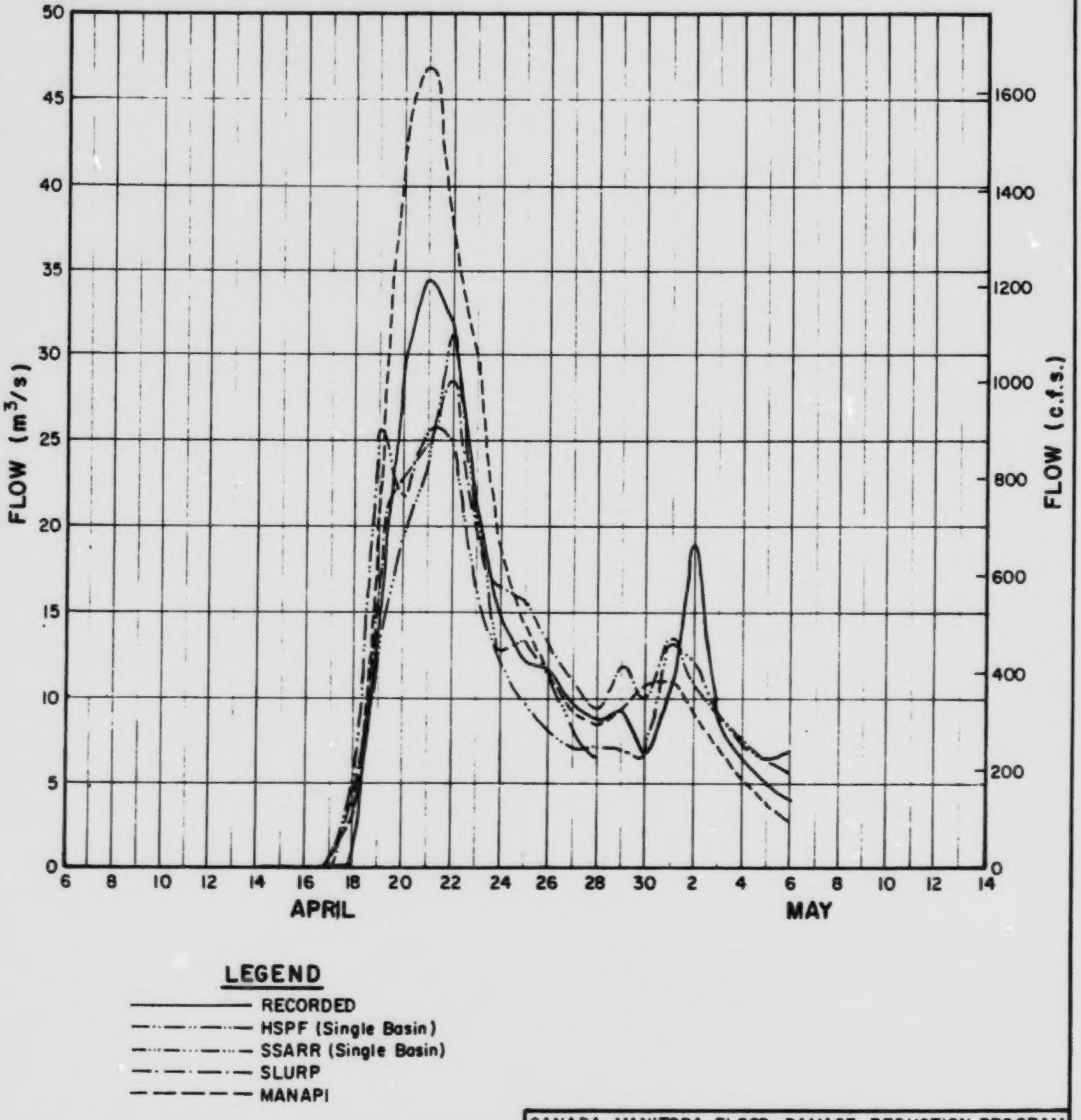
DEPARTMENT OF NATURAL RESOURCES

**WATER RESOURCES BRANCH**

## SIMULATED AND RECORDED FLOWS
## ROSEISLE CREEK NEAR ROSEISLE
## CALIBRATION – SPRING OF 1976

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| PREPARED E.L.B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

**FIGURE E-11**



CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA

DEPARTMENT OF NATURAL RESOURCES

WATER RESOURCES BRANCH

## SIMULATED AND RECORDED FLOWS ROSEISLE CREEK NEAR ROSEISLE

## CALIBRATION - SPRING OF 1979

| | | | | | | | |
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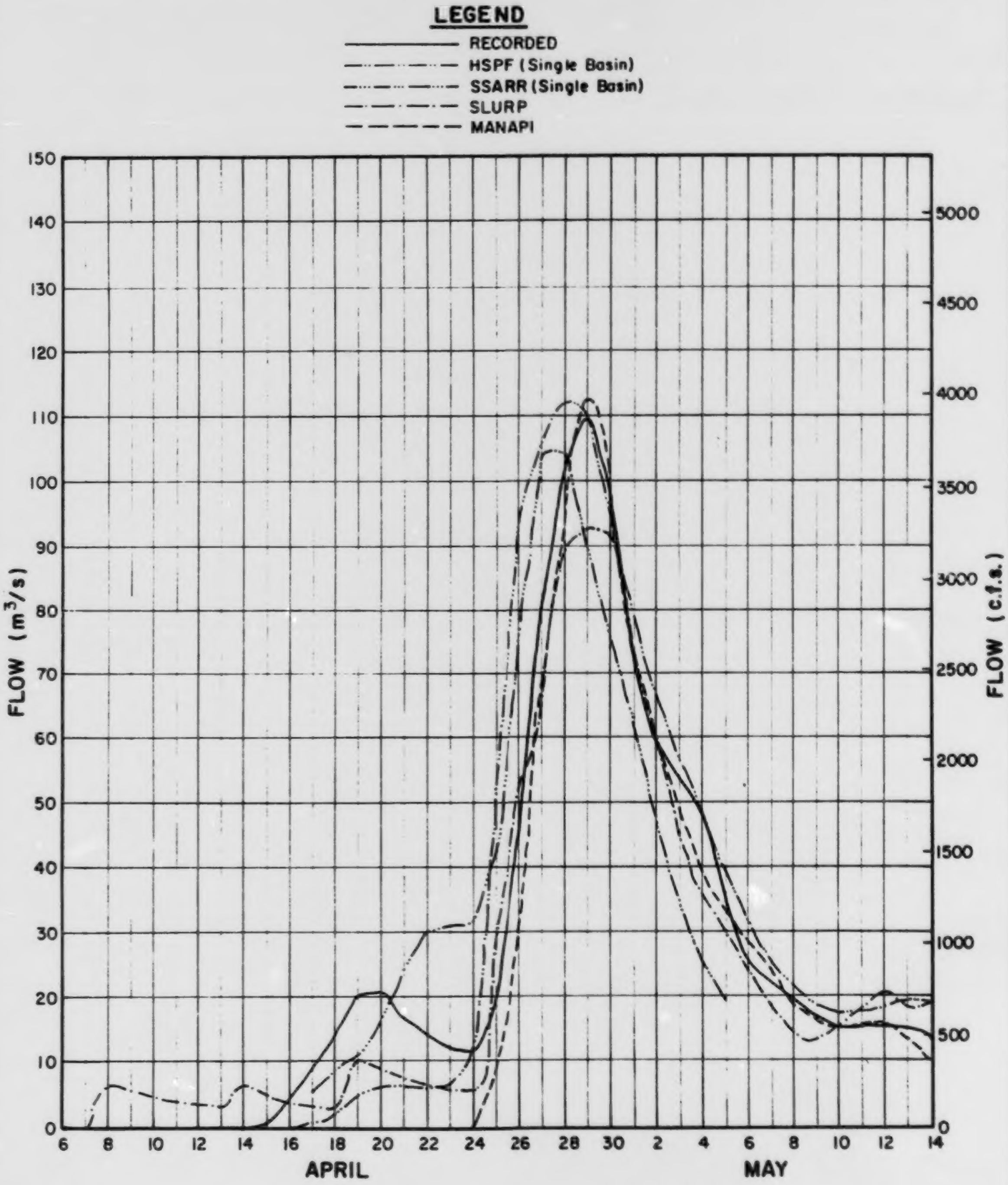
| PREPARED<br>E. L. B. | DRAWN C.E.W.<br>CHECKED A.W. | SUBMITTED | APPROVED | SCALE AS SHOWN | DATE 84 05 22 | SHEET OF | FILE NO. |

FIGURE E-12



CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

## SIMULATED AND RECORDED FLOWS
## BOYNE RIVER NEAR STEPHENFIELD
## CALIBRATION - SPRING OF 1970

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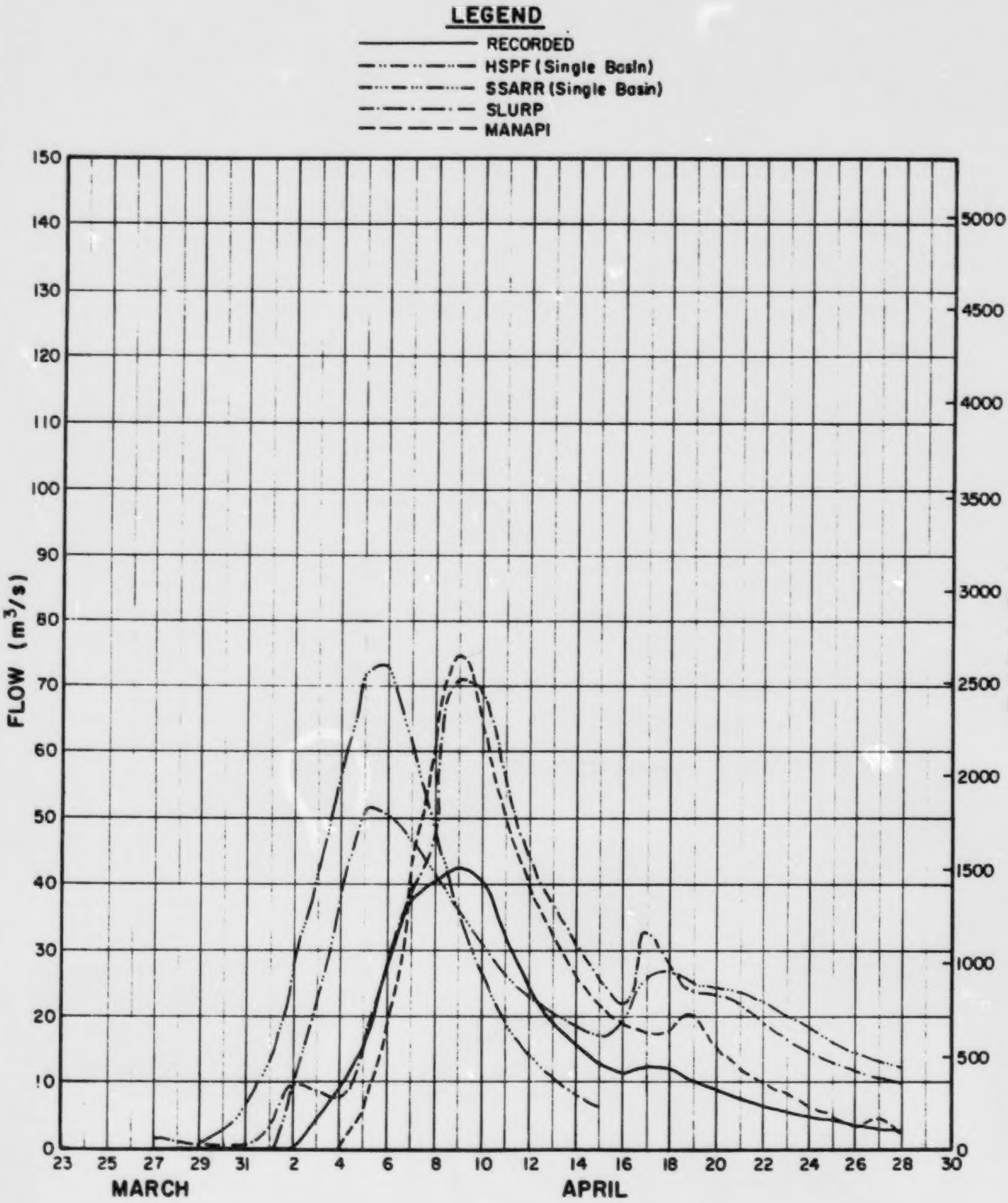
FIGURE E-13



CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
**WATER RESOURCES BRANCH**

## SIMULATED AND RECORDED FLOWS BOYNE RIVER NEAR STEPHENFIELD
### CALIBRATION - SPRING OF 1976

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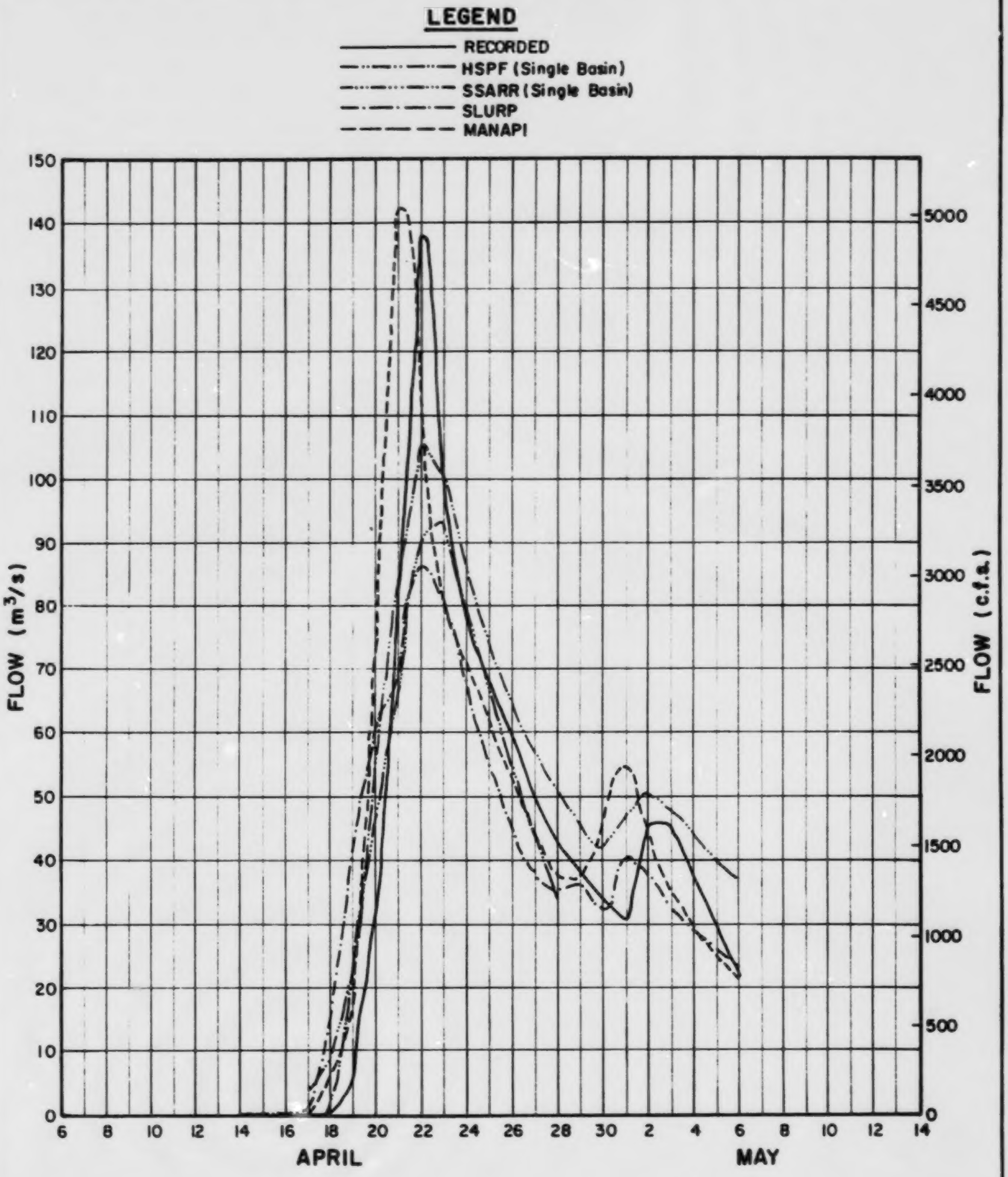
FIGURE E-14



CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE ONE

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
WATER RESOURCES BRANCH

## SIMULATED AND RECORDED FLOWS BOYNE RIVER NEAR STEPHENFIELD CALIBRATION - SPRING OF 1979

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FIGURE E-15

## Appendix F

# CHANNEL ROUTING MODELS

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## F.1 INTRODUCTION

Muskingum flood routing is the method currently used on major river channels by the Manitoba Water Resources Branch for operational flood forecasting. Unit hydrographs are used for conveyance on tributaries. As such it was the standard against which the performance of the other routing models was judged.

## F.2 MUSKINGUM ROUTING

Muskingum routing (McCarthy, 1938) is a simple technique which concentrates on the concept of storage of the flood water and does not directly include the effects of resistance to the flow. The data requirements are the absolute minimum required for the application of any routing method, consisting simply of upstream and downstream hydrographs of observed floods. Routing coefficients for each reach are calculated from the routing parameters K and X. K is the basic storage parameter and X relates the relative importance of inflow and outflow in obtaining an index of storage in the reach.

Initial estimates of the routing parameters for a specific event can be obtained by graphical analysis of the observed hydrographs or can be based on the judgement of an experienced hydrologist. During calibration, adjustments are made to the routing parameters until the routed hydrograph most closely resembles the observed hydrograph at the downstream station. Once this has been done for several events of different magnitudes a table or graph can be constructed to indicate the appropriate parameters to be used in routing the flood under consideration.

There are two serious limitations to the application of the Muskingum method. Firstly, the routing parameters remain constant throughout the passage of a flood event, which in some cases can lead to serious errors. Secondly, routing of a flood with a magnitude outside the calibrated range can lead to serious errors. The second limitation is common to all routing techniques but, since the Muskingum method does not use the physical characteristics of the channel or the flow directly in determining the routing parameters, there is a greater chance of serious errors.

## F.3 MUSKINGUM-CUNGE ROUTING

Muskingum-Cunge (Cunge, 1969) was devised to improve the basic Muskingum method by utilizing the physical characteristics of the channel to determine the routing parameters. Cunge defined the parameters for Muskingum in terms of the parameters for the more sophisticated diffusion method of routing although the basic routing formula remained unchanged. The routing coefficients incorporate an attenuation parameter, which is based upon channel characteristics. This allows the attenuation of the flood peak to be determined with a fair amount of precision.

The formula used to calculate the attenuation parameter requires information on the flooded area, time of travel through the reach, channel slope and channel length for several flood events. It is preferable if these events do not include large amounts of lateral inflow between stations. Graphical relationships between peak upstream discharge versus the speed of the flood wave and the attenuation parameter are then derived in determining the appropriate routing parameters for the range of observed streamflow events.

The computer version of this model allows for the addition of up to two tributaries as well as a distributed lateral inflow. It also adjusts the routing coefficients internally when changes in routing parameters or the modelling period are made.

This method still does not allow for changes in the routing parameters during the course of a flood event.

## F.4 SSARR MODEL ROUTING

Internal routing procedures of the SSARR watershed model provide for changes in routing parameters during the passage of a flood, albeit in a relatively simple way. SSARR routing utilizes the concept of time of storage in a series of reservoirs or phases to account for the channel's resistance to flow. As the resistance to flow is not constant during the passage of a flood, this method allows the time of storage to be varied with discharge. The relationship between time of storage and discharge can be given in a table or defined by a mathematical equation.

The routing parameters are determined from the reach length and the time required for floods of varying peak magnitude to travel through the reach. These are used to define the number of subreaches or phases for the first trial and the time of storage within each phase for a given discharge. During calibration, adjustments are made to the number of phases and the time of storage until the routed hydrograph is an accurate representation of the observed downstream hydrograph.

Once the routing parameters have been calibrated it is not necessary to make further adjustments unless the flood event to be routed is outside the calibration range.

## F.5 HSPF MODEL ROUTING

Internal routing procedures of the HSPF watershed model incorporate both the physical characteristics of the channel and variable resistance to the flow to define the routing parameters. The reach is divided into subreaches of equal length, which are sufficiently short so that the flow rate throughout each subreach can be considered constant. Tables are then constructed which relate stage to storage volume, flooded area and discharge for each of the subreaches. This, of course, requires that sufficient cross-section and rating curve data be available so that the information contained in the tables will be reasonably accurate. If rating curves are not available they can be created using cross-section information and the Mannings formula.

If the physical characteristics are accurately defined, calibration is accomplished solely by adjusting a factor which weights inflows and outflows from the subreaches.

## F.6 ONE-DIMENSIONAL DYNAMIC ROUTING

The One-Dimensional Model (M. Sydor, pers. comm.) is a comprehensive flood routing method employing the St. Venant equations for conservation of mass, continuity and conservation of momentum. As such it has the capacity for computing water levels, discharges and flow velocities at any point along a reach. Variations in channel geometry and resistance, as well as control structures, can be incorporated into the routing. It also has the capacity to compute the variable backwater effects due to tributary inflows.

The major limitations to the model for small streams are its data requirements and its sensitivity. Use of the model requires information on the channel cross-section, the portion of the channel actively conveying water and channel resistance coefficients. To accurately calibrate the model, water levels along the reach corresponding to specific discharges are also required. Because of the numerical techniques used to solve the equations, the routing configuration and modelling period must be carefully chosen so as not to create a computational instability. If initial parameter estimates are not carefully selected, or the changes in slope or cross-section are too rapid, it is possible to achieve a zero storage in a reach of the channel causing the simulation to fail.

The risk of failure of the procedure is less in large rivers where the changes in cross-section tend to be less abrupt and where the volume of water being conveyed is greater, providing more allowable error in initial parameter estimates.

## F.7 CONCLUSIONS

Rigorous attempts at calibration were made using only the One-Dimensional and Muskingum-Cunge routing procedures.

The One-Dimensional model proved overly sensitive to the changes in stream channel characteristics on the Boyne River. This resulted in an increase in the number of computational sections, a reduction in the modelling interval and an increase in modelling costs to the point where this method was not practical.

Muskingum-Cunge showed promise but was difficult to calibrate when large amounts of lateral inflows produced uncertainties in the mainstream flows between gauging points. Whether its increased flexibility and theoretical superiority over the basic Muskingum will justify the additional data requirements and time required for set-up has yet to be determined.

Routing models were not fully compared during Phase One. The performance and suitability of the four non-dynamic routing procedures will be compared during Phase Two. Dynamic routing will be attempted on the main stem of the Red River from the United States boundary to Winnipeg, where the relatively shallow river slope and the significant backwater effects appear to meet the conditions required for the effective use of this model.

## F.8 LITERATURE CITED

1. Cunge J.A. (1969) On the subject of a flood wave propagation method. Journal of Hydraulics Research, IAHR, 7, 205-230.
2. McCarthy G.T. (1938) The unit hydrograph and flood routing. Unpublished manuscript presented at a conference of the North Atlantic Division, U.S. Army Corps of Engineers, 24 June.

## Appendix G

# REAL-TIME DATA ACQUISITION

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# ANNEX'S

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## G.1 INTRODUCTION

Four real-time hydrometeorological data collection stations with electronic data transmission capabilities were installed as part of Phase One studies under The Canada-Manitoba Flood Forecasting Agreement. This appendix describes these instruments and the data transmission facilities and compares them with respect to their basic features, performance and cost. The value of these systems for flood forecasting is described in the main report.

Four existing hydrometric stations in Manitoba were equipped with two different types of real-time hydrometeorological data collection instrumentation for testing and evaluation purposes. The systems are similar in nature, differing mainly in how the user accesses the data.

With the Data Collection Platform (DCP) System, the DCP transmits the data to a Geostationary Satellite. The data is then retransmitted to a receive site in Washington D.C. The user then accesses the data from a storage and dissemination facility at the receive site via a dial-in port.

With the Data Acquisition and Telemetry System (DATS), a land line based real-time system is employed. With this system the user accesses the data directly from the station over a telephone line using a data terminal.

## G.2 FIELD INSTRUMENTS

### G.2.1 Data Collection Platforms (DCP's)

DCP's manufactured by Bristol Aerospace were installed at Birdtail Creek near Birtle and at Stephenfield Reservoir. The location of these stations is shown on Figure G-1. The DCP System consists of:

- electronics package with a radio transmitter
- digital encoder interfaced to an analogue water level sensor
- directional antenna
- solar panel and 12 volt rechargeable battery

Depending on the sensor configuration, each of these DCP's is capable of collecting and transmitting data on a number of hydrological and meteorological parameters. In addition to a water level sensor the Birdtail Creek station was also equipped with a tipping bucket rain gauge in order to monitor rainfall intensity on a real-time basis. The DCP transmits data every three hours in a predetermined one minute time slot via a directional antenna to the GEOS East Geostationary satellite. Each transmission includes data that has been archived at various sampling intervals, normally one hour. The data is then retransmitted to a ground receive site in Washington, D.C., where it can be accessed by the user with a data terminal. The DCP system is powered by a 12-volt battery which is recharged by a solar panel, making it suitable for remote locations where electrical power is not available. The instruments for the DCP station on the Stephenfield Reservoir are depicting on Figure G-2. Details on the DCP equipment and its performance are described in Annex G-1.

### G.2.2 Data Acquisition and Telemetry System (DATS)

DATS systems manufactured by Canadian Applied Technology (CAT) of Toronto were installed on the Hespeler Floodway and on the Boyne River near Roseisle. The location of these stations is shown on Figure G-1. The DATS system consists of:

- electronics package complete with communication module
- incremental shaft encoder interfaced with a water level sensor
- auto answer telephone modem
- battery charger and 12-volt battery

AC power and telephone service are required for the system to function. The unit is remotely accessible and programmable over the telephone lines using a standard data terminal. Parameters such as sampling rate, time, water level, station identifier, etc., can be changed using the terminal, by making a telephone call to a telephone interface and thence to the DATS at the station. On site memory capacity for a standard DATS unit is one month of hourly water telephone call to a telephone interface and thence to the DATS at the station. On site memory capacity for a standard DATS unit is one month of hourly water level readings. The larger DATS units have the capability of storing one week of hourly values from a variety of hydrologic and meteorologic sensors. In addition to the water level sensor, the station on the Boyne River at Roseisle is also equipped with a Belfort Type weighing precipitation gauge which makes it possible to monitor total precipitation and to retrieve it on a real-time basis from the site. The instruments for the DATS station on the Boyne River near Roseisle are depicted on Figure G-3. Details on the DATS equipment and its performance are described in Annex G-2.

## G.3 DATA TRANSMISSION AND DECODING

### G.3.1 DCP

Data is obtained by initiating a telephone call to the data storage and dissemination facility over conventional telephone lines using a data terminal. The output contains system diagnostic (time, signal strength, frequency drift, etc.) as well as sensor data that must be decoded to put it in a form that is meaningful to the user. The ideal data processing system includes an automatic dialing system initiated by a computer to access the data storage facilities. A computer program then reformats the data and stores it in the user's computer files. This type of system is used by the Flood Forecast Centre to access the data from DCP's in Manitoba. A schematic of this data transmission system is shown on Figure G-4.

Data storage capabilities at the receive site are limited to one day of hourly values. Therefore the data must be retrieved daily to prevent a loss of record.

### G.3.2 DATS

Data is obtained by initiating a telephone call to the on-site unit over conventional telephone lines using a data terminal. The output is user friendly and does not require decoding. All information necessary to make the output meaningful to the user is contained on the output. Data from the DATS units are obtained via the automated program that the Flood Forecast Centre developed to access the satellite data storage and dissemination facilities. An advantage with the DATS system is the on-site memory capacity of up to 30 days of hourly water level values. A further advantage is that it can be programmed remotely to obtain data as frequently as one minute. A schematic of this data transmission system is shown on Figure G-4.

## G.4 BENEFIT/COST CONSIDERATIONS OF DCP VS DATS

### G.4.1 Reliability and Suitability

Both DCP and DATS systems have many advantages and disadvantages relating to the performance and reliability of the instruments and the data transmission methodologies, the type of data available and the cost of using the system. The more important advantages and disadvantages of each system are shown in Annex G-3.

A tentative conclusion based on a few years use of these instruments is that the instruments used with the DCP system have had a better performance record than the instruments used with the DATS system. Also satellite data transmission has been more reliable than the land-line communication used by the DATS system.

### G.4.2 Capital Cost and Operating Cost

Data on capital costs and operating costs, in 1983 Canadian dollars, are shown in Annex G-4 for both DCP and DATS systems. The capital cost of the DCP is higher, but the operating cost is so much lower that the total cost of the DCP would be less by the end of the third year of operation.

## ANNEX G-1
## Data Collection Platform (DCP) Equipment

### Stephenfield Reservoir

A. Equipment:

- Bristol GOES D.C.P. Model 696-07-02-1200
- Synergetics Model 18B GOES Crossed Yagi Antenna
- Stevens Memomark II Digital Encoder actuated by a U.S.G.S. Water Stage Manometer
- Solarex 5 Watt 12-Volt Nominal Unipanel
- Varta 24RV-400 80 amp. hour 12-volt lead acid battery

B. Performance:

The unit was installed in September, 1982. The system has operated satisfactorily since that time with the following exceptions:

- five days of water level readings lost due to intermittent malfunction of the digital encoder
- three days of transmissions lost due to ice accretion on the directional antenna

### Birdtail Creek near Birtle

A. Equipment:

- Bristol GOES DCP Model 696-07-02-1200
- Synergetics Model 18B GOES Crossed Yagi Antenna
- Stevens Memomark II Digital Encoder (Float Actuated)
- Atmospheric Environment Service Tipping Bucket Rain Gauge
- Solarex 5 Watt 12-Volt Nominal Unipanel
- Varta 24RV-400 80 amp. hour lead acid battery

B. Performance:

The unit was installed in March, 1983. The unit has performed satisfactorily since that time with the following exceptions:

- The tipping bucket rain gauge mercury switch was replaced in June 1983 due to an intermittent malfunction.
- Three weeks of water level readings were lost due to malfunctioning digital encoder.

# ANNEX G-2

## Data Acquisition and Telemetry System (DATS) Instruments

### Boyne River near Roseisle

A. Equipment:

- Canadian Applied Technology (CAT) SN2200 DATS Unit
- CAT Incremental Shaft Encoder activated by a CAE Model 311M Servomanometer
- Belfort Weighing Type Precipitation Gauge
- 103A2R Auto Answer Telephone Modem
- CAT Battery Charger
- Varta 24RV-400 80 amp. hour 12-volt lead acid battery

B. Performance:

The instrumentation was originally installed Sept. 30, 1982. A number of problems have affected the performance of the system since installation:

- A system grounding problem between the DATS unit, battery charger, Belfort Rain Gauge and Modem caused considerable loss of real-time record.
- The system is susceptible to static charge buildup as well as surges in the AC power input lines.
- The unit operated satisfactorily from June 1 to late August. A summer storm with resulting electrical activity damaged the power supply and DATS circuits which resulted in the total failure of the system. The unit was returned to the manufacturer and was reinstalled January 1, 1984.

### Hespeler Floodway near Rosenfeld

A. Equipment:

- CAT Model SH2200 DATS Unit
- CAT Incremental Shaft Encoder actuated by a CAE Model 311M Servomanometer
- 103A2R Auto Answer Telephone Modem
- CAT Battery Charger
- Varta 24RV-400 80 amp. hour 12-volt lead acid battery

B. Performance:

The unit was installed in October, 1982. The system has operated satisfactorily with a number of exceptions:

- During the open water period in 1983 intermittent telephone line service prevented users from accessing the data for periods of one to two days at various times.
- The problems relating to telephone service appear to be a function of the quality of the telephone exchange in the area
- four days of record were lost in May, 1983 due to a power outage at the station.

# ANNEX G-3
## Reliability and Suitability - DCP VS DATS

### DCP Advantages

- percentage of messages received is over 99%
- system has low power consumption and and can be powered by batteries and solar panels
- designed to operate unattended for long periods of time and suitable for remote operations
- operates to -40° without requiring external heat

### DCP Disadvantages

- outside agency operates receive site, data storage and dissemination facilities
- user fees for the use of satellite and data storage and dissemination facilities may be introduced at some future time (costs are unknown
- difficult to access data at peak periods due to overloading of dial in facilities at receive site
- storage at the data storage and dissemination facility is limited to 24 hours
- cost of accessing the data storage and dissemination facility via phone lines is high
- user must develop software to reformat output
- programming changes for sampling rate time, etc., must be done on site
- data is available at intervals of three hours or more

### DATS Advantages

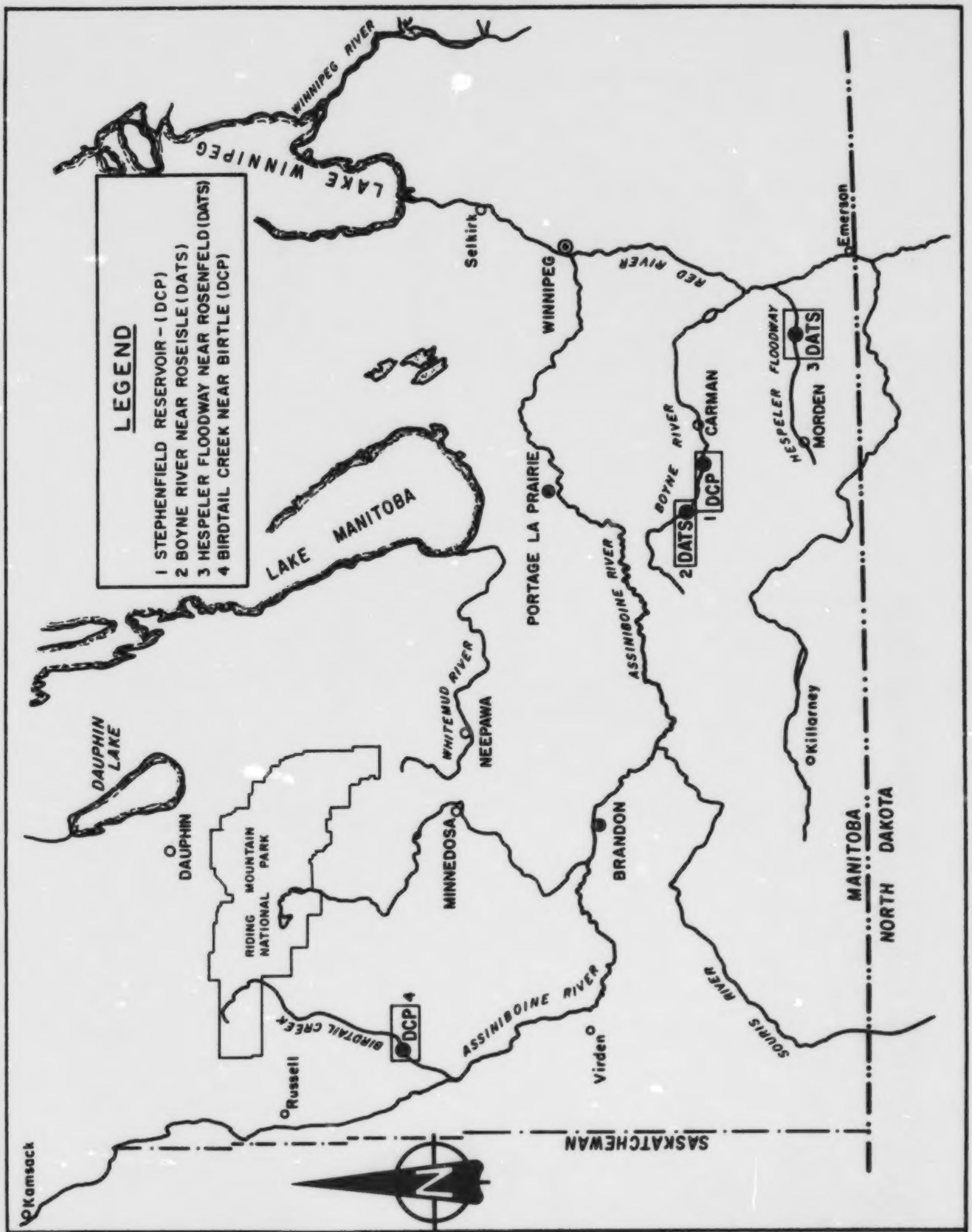
- data can be accessed directly from the station by user
- output is "user friendly" (all necessary information to make the data meaningful to user is included in the output)
- on site data storage up to 30 days of hourly data
- real-time data can be obtained at any time

### DATS Disadvantages

- dependent on telephone and AC power service for operation
- the present units have relatively high power requirements
- expensive to upgrade existing sites for power and telephone
- telephone lines are unusually susceptible to failure during periods of high run-off (usually when the data is needed the most)

## ANNEX G-4
## Capital Cost and Operating Cost

| DCP Capital Cost | | DATS Capital Cost | |
|---|---|---|---|
| DCP | $ 4,600.00 | DATS | $ 4,500.00 |
| Digital Encoder | 1,500.00 | Battery | 100.00 |
| Solar Panels | 350.00 | AC Surge Protection | 350.00 |
| Battery | 100.00 | TOTAL | $ 4,950.00 |
| TOTAL | $ 6,550.00 | | |
| (Does not include cost of programming set = $700.00. Usually one set is required for every 7-10 units). | | (Does not include power and telephone hook-up charges). | |
| Annual Operating Cost: | $50.00 | Annual Operating Cost: | |
| (Assumes no user fees for the use of the system and does not include telephone or computer charges). | | Telephone (line- and Modem rental) | $ 300.00 |
| | | Power | 400.00 |
| | | TOTAL | $ 700.00 |
| | | (Does not include computer costs). | |


LEGEND
1 STEPHENFIELD RESERVOIR - (DCP)
2 BOYNE RIVER NEAR ROSEISLE (DATS)
3 HESPELER FLOODWAY NEAR ROSENFELD (DATS)
4 BIRDTAIL CREEK NEAR BIRTLE (DCP)
WINNIPEG RIVER
LAKE WINNIPEG
Selkirk
WINNIPEG
RED RIVER
Emerson
LAKE MANITOBA
PORTAGE LA PRAIRIE
BOYNE RIVER
CARMAN
1 DCP
2 DATS
HESPELER FLOODWAY
MORDEN
3 DATS
ASSINIBOINE RIVER
WHITEMUD RIVER
NEEPAWA
DAUPHIN LAKE
DAUPHIN
RIDING MOUNTAIN NATIONAL PARK
MINNEDOSA
BRANDON
Killarney
MANITOBA
NORTH DAKOTA
SOURIS RIVER
BIRDTAIL CREEK
DCP 4
Virden
Russell
Kamsack
SASKATCHEWAN
N

PROVINCE OF MANITOBA
DEPARTMENT OF NATURAL RESOURCES
**WATER RESOURCES BRANCH**

## LOCATION OF REAL - TIME HYDROMETEOROLOGICAL DATA STATIONS

| PREPARED | DRAWN | SUBMITTED | APPROVED | SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|---|---|---|---|
| A. Workentin | F. R.<br>CHECKED | | | AS SHOWN | 85 02 11 | 1 OF 1 | |

MG 7561

FIGURE G-1

# STEPHENFIELD RESERVOIR
# DATA COLLECTION PLATFORM

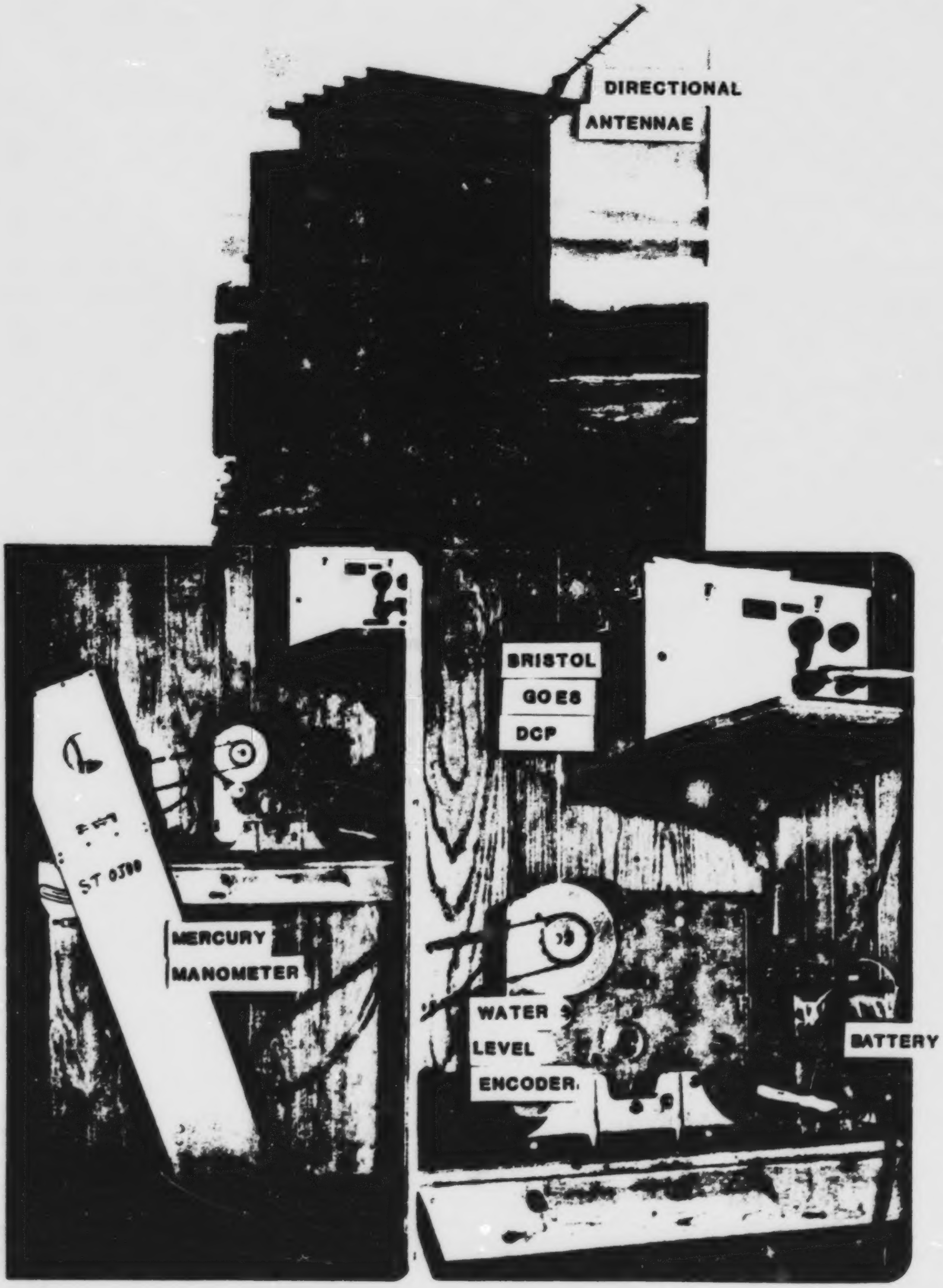

DIRECTIONAL
ANTENNAE
BRISTOL
GOES
DCP
ST 0300
MERCURY
MANOMETER
WATER
LEVEL
ENCODER
BATTERY

FIGURE G-2

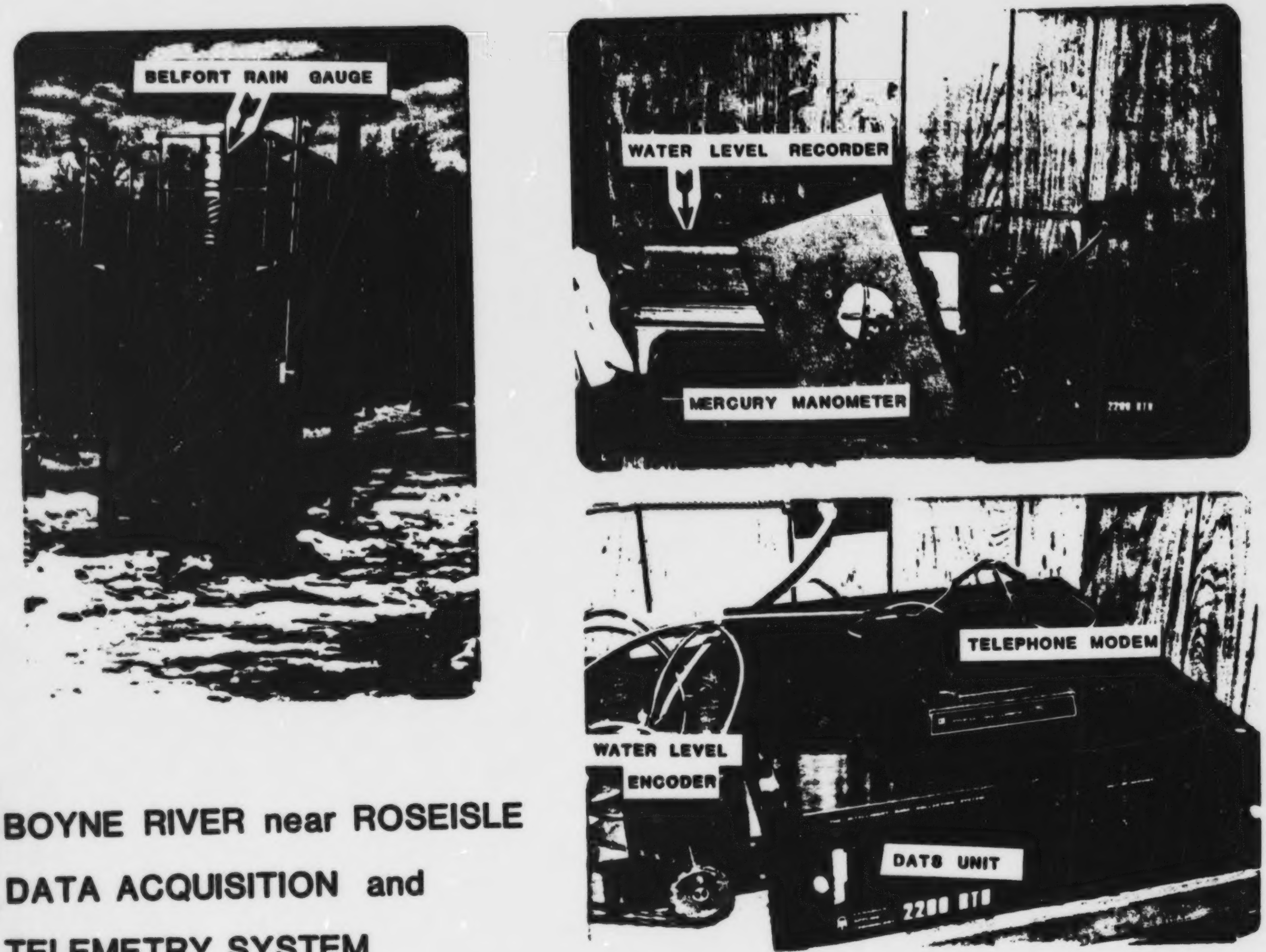

BELFORT RAIN GAUGE
WATER LEVEL RECORDER
MERCURY MANOMETER
TELEPHONE MODEM
WATER LEVEL ENCODER
DATS UNIT
2200 RTU

FIGURE G-3

BOYNE RIVER near ROSEISLE
DATA ACQUISITION and
TELEMETRY SYSTEM

# DCP AND DATS
# DATA TRANSMISSION SYSTEMS

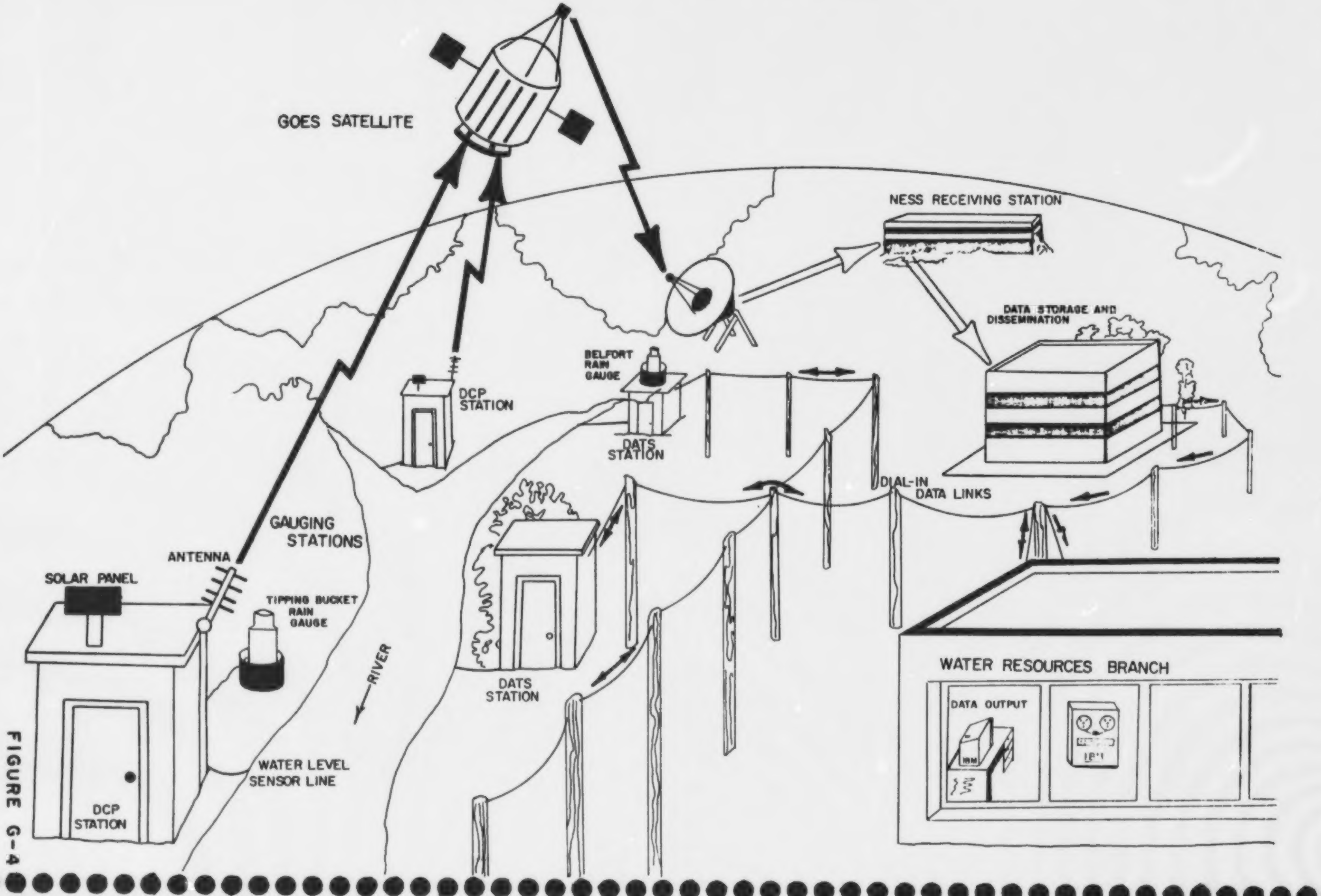

GOES SATELLITE
NESS RECEIVING STATION
DATA STORAGE AND DISSEMINATION
BELFORT RAIN GAUGE
DCP STATION
DATS STATION
DIAL-IN DATA LINKS
GAUGING STATIONS
ANTENNA
SOLAR PANEL
TIPPING BUCKET RAIN GAUGE
RIVER
DATS STATION
WATER RESOURCES BRANCH
DATA OUTPUT
IBM
WATER LEVEL SENSOR LINE
DCP STATION

FIGURE G-4

## Appendix H

# CANADA - MANITOBA AGREEMENT RESPECTING FLOOD FORECASTING

# AN AGREEMENT RESPECTING FLOOD FORECASTING

THIS AGREEMENT made this 31st day of March, 1981

BETWEEN:

THE GOVERNMENT OF CANADA

hereinafter referred to as "Canada"

OF THE FIRST PART

AND

THE GOVERNMENT OF MANITOBA

hereinafter referred to as "the Province"

OF THE SECOND PART

WHEREAS

(1) on the 20th day of December, 1976, the Parties signed both a General Agreement which sets out policies for reducing potential flood damages and an Agreement for Flood Risk Mapping;

(2) the said General Agreement, in sub-clause 3(2) thereof, provides that the Parties hereto from time to time may enter into other agreements, including agreements respecting flood forecasting systems;

(3) by Order-in-Council No. PC1980-4/2253 made the 27th day of August, 1980, the Governor in Council has authorized the Minister of the Environment for Canada to enter into this Agreement on behalf of Canada; and

(4) by Order-in-Council No. 69/81 made the 21st day of January, 1981, the Lieutenant Governor in Council has authorized the Minister of Natural Resources for Manitoba to enter into this Agreement on behalf of the Province.

IT IS THEREFORE AGREED BETWEEN THE PARTIES HERETO AS FOLLOWS:

## DEFINITIONS

1. In this Agreement,

(a) "General Agreement" means the General Agreement Respecting Flood Damage Reduction signed by the Parties on the 20th day of December, 1976; and

(b) "Steering Committee" means the Committee established pursuant to the "General Agreement" of the 20th day of December, 1976.

## FLOOD FORECASTING

2. Objective

The objective of this Agreement is to provide for the development or improvement of flood forecasting capabilities for the Red, Assiniboine and Souris Rivers in Manitoba that are of interest to both Canada and the Province. This Agreement shall be carried out in two phases as described below.

(a) Phase One: Planning and Design

The Parties shall, in cooperation with each other and as hereinafter provided, carry out the following:

(i) carry out a pilot project on the Boyne River for the purpose of developing a suitable deterministic runoff model for use on the Red, Assiniboine and Souris Rivers in Manitoba,

(ii) evaluate the need for, and where necessary, arrange for installation of appropriate equipment to obtain additional field data required for carrying out the Boyne River pilot project,

(iii) carry out such tasks as may be required to design, develop, calibrate, test and prepare, and document formal procedures for an adequate flood forecasting system for the Red, Assiniboine and Souris Rivers of Manitoba,

(iv) establish and staff a flood forecasting centre to be located in the offices of the Manitoba Department of Natural Resources for the Red, Assiniboine and Souris Rivers in Manitoba, and

(v) undertake tasks related or necessarily incidental to the above.

(b) Phase Two: Development and Operation

Upon conclusion of Phase One, the Parties shall decide whether to proceed with Phase Two based on a recommendation and report from the Steering Committee. The Parties shall then, in cooperation with each other and as hereinafter provided, carry out the following activities:

(i) operate the Flood Forecasting Centre and the information networks to provide at all times the best practicable flood forecasts for the Red, Assiniboine and Souris Rivers in Manitoba,

(ii) evaluate the need for, and where necessary, arrange for installation of appropriate equipment to obtain additional field data required to produce an effective and adequate flood forecasting system for the Red, Assiniboine and Souris Rivers in Manitoba,

(iii) acquire, or lease, and install field data instruments, data transmission facilities, and office equipment required to effectively operate a flood forecasting centre for the Red, Assiniboine and Souris Rivers in Manitoba,

(iv) carry out such other tasks as may be required to further design, develop, calibrate and test an effective and adequate flood forecasting system for the Red, Assiniboine and Souris Rivers in Manitoba, and

(v) undertake tasks related or necessarily incidental to the above.

The Parties agree that during Phase Two the Manitoba Department of Natural Resources shall be responsible for issuing flood warnings based on the forecasts provided by the Flood Forecasting Centre.

(c) A Party may, with the approval of the Steering Committee, individually enter into a contract to have any Phase One or Phase Two work done by a third party.

(d) Study reports issued by the Flood Forecasting Centre shall bear an appropriate acknowledgement of the role of both Parties.

## STEERING COMMITTEE

3. The Steering Committee shall:

    (a) be responsible for the general administration of this Agreement;

    (b) determine how responsibilities for the work programs described in clause 2 will be divided;

    (c) establish the financial procedures referred to in clause 9;

    (d) keep minutes of its meetings;

    (e) review and, if satisfactory, approve any proposal that a contract be entered into by either Party;

    (f) review any work programs prepared by the Technical Sub-Committee, hereinafter provided in clause 4, for the approval of the Parties and for preparing an annual budget;

    (g) establish a Technical Sub-Committee on Flood Forecasting and various other sub-committees as necessary to assist in meeting the objective

    (h) carry out such other related duties as the Parties may request.

## TECHNICAL SUB-COMMITTEE

4. The Technical Sub-Committee on Flood Forecasting shall be responsible for:

    (a) preparing and recommending to the Steering Committee on or before the 31st day of December in each year of this Agreement, work programs for the following year commencing the 1st of April;

    (b) ensuring that the work programs are carried out properly and on time;

(c) recommending to the Steering Committee new ideas and applications that relate to the betterment of the forecast and the operational aspects of the Flood Forecasting Centre;

(d) liaison with other agencies with involvement or interests in flood forecasting, including agencies in Saskatchewan and the United States; and

(e) carrying out such other related duties as the Steering Committee may request.

## FINANCES

5(1) Subject to sub-clause (2), the costs of carrying out this Agreement shall be shared equally by the Parties but the total sum payable by each Party for such costs under this Agreement shall not exceed $300,000;

(a) in addition, the total costs of carrying out Phase One, including the Boyne River pilot project provided in sub-clause (1)(b), shall not exceed $350,000; and

(b) the total costs of carrying out the Boyne River pilot project shall not exceed $190,000.

(2) Each Party shall bear the entire cost of the salary and travelling and related expenses of:

(a) each of its members of the Steering Committee and Technical Sub-Committee, and of any other sub-committee established pursuant to sub-clause 3(g);

(b) each of its employees who, although engaged in an activity hereunder, does not have a specified portion of his work day, week, month or year assigned exclusively to the carrying out of this Agreement; and

(c) each of its employees who, prior to the signing of this Agreement was engaged in activities, related to flood forecasting that would have continued even without this Agreement.

6(1) Where a Party supplies, for the purpose of carrying out this Agreement:

(a) material, equipment or other property;

(b) the use of office space or any facilities;

(c) the operation of hydrometric and meteorological stations specifically for forecasting floods, and incremental costs of getting real-time data from regular stations; or

(d) services, other than the services referred to in sub-clause 5(2); the same shall be supplied at cost.

(2) Unless otherwise specified by the Steering Committee:

(a) upon termination of this Agreement, ownership of special equipment installed at hydrometric and meteorological stations shall be vested in Canada; and

(b) upon termination of this Agreement, ownership of special equipment installed in the Flood Forecasting Centre shall be vested in the Province.

7. The Parties agree that the portion of the hydrometric network which is key to the successful completion of the Agreement, either existing at the start of the Agreement or installed as part of the Agreement itself, shall continue in operation during the duration of the Agreement. Classification of those key stations existing at the start of the Agreement shall remain in accordance with Schedule A of the Memorandum of Agreement for Hydrometric Surveys. Additional hydrometric stations established for the purposes of this Agreement shall be installed and operated as provided in clause 2(b)(ii). Upon termination of the Agreement additional hydrometric stations established under the Agreement shall be classified federal-

provincial and be operated under the Memorandum of Agreement for Hydrometric Surveys. Canada agrees that hydrometric real time data acquisition equipment installed as part of the Agreement shall be operated at Canada's expense for an additional five years beyond the termination date of this Agreement.

8. The Province agrees to continue the operation of the flood forecasting system developed for the Red, Assiniboine and Souris Rivers in Manitoba pursuant to clause 2 of this Agreement with any subsequent improvements for an additional five years beyond the termination date of this Agreement.

9(1) The Province shall, with respect to the costs that are covered by this Agreement pay such costs as they become due, and shall submit progress claims to Canada on a quarterly basis, setting out the eligible costs actually incurred and paid.

(2) Canada shall, upon receipt of claims from the Province, promptly reimburse the Province for its share of the costs.

(3) Where eligible costs are incurred and paid for by Canada, the Province shall reimburse Canada promptly upon receipt of invoices which Canada shall submit quarterly.

10(1) Each Party shall maintain adequate documentation and records of the costs that are to be shared by the Parties and which are incurred by it and shall, upon request, make available such documentation and records for examination by auditors of the other Party.

(2) Any discrepancy disclosed by audit, in the amount paid by Canada to the Province pursuant to sub-clause 9(2) and in the amount paid by the Province to Canada pursuant to sub-clause 9(3) shall be promptly rectified by and adjusted between the Parties.

## GENERAL

11(1) Subject to sub-clause (2),

(a) Phase One of this Agreement shall terminate no later than the 31st day of March, 1983;

(b) Phase Two of this Agreement shall terminate no later than the 31st day of March, 1985, if the Parties decide to proceed with this Phase upon conclusion of Phase One; and

(c) either Party may terminate this Agreement by advising the other Party at least six months prior to the desired termination date.

(2) The provisions of this Agreement respecting the payment of costs that are to be shared by the Parties shall remain in effect for one year beyond the termination date set out in or agreed to pursuant to sub-clause (1).

12. No member of the Parliament of Canada or of the Legislative Assembly of the Province shall hold, enjoy or be admitted to any share, part or benefit from this Agreement or any agreement, contract or benefit arising therefrom.

IN WITNESS WHEREOF the Honourable John Roberts, Minister of the Environment for Canada, has hereunto set his hand on behalf of Canada and the Honourable Harry J. Enns has hereunto set his hand on behalf of the Province.

SIGNED by the Honourable John Roberts the Minister of the Environment on behalf of the Government of Canada,

"John Roberts"
Minister

In the Presence of

"Nickolas P. Heseltine"
WITNESS

SIGNED by the Honourable Harry J. Enns the Minister of Natural Resources on behalf of the Government of the Province of Manitoba,

"Harry J. Enns"
Minister

In the Presence of

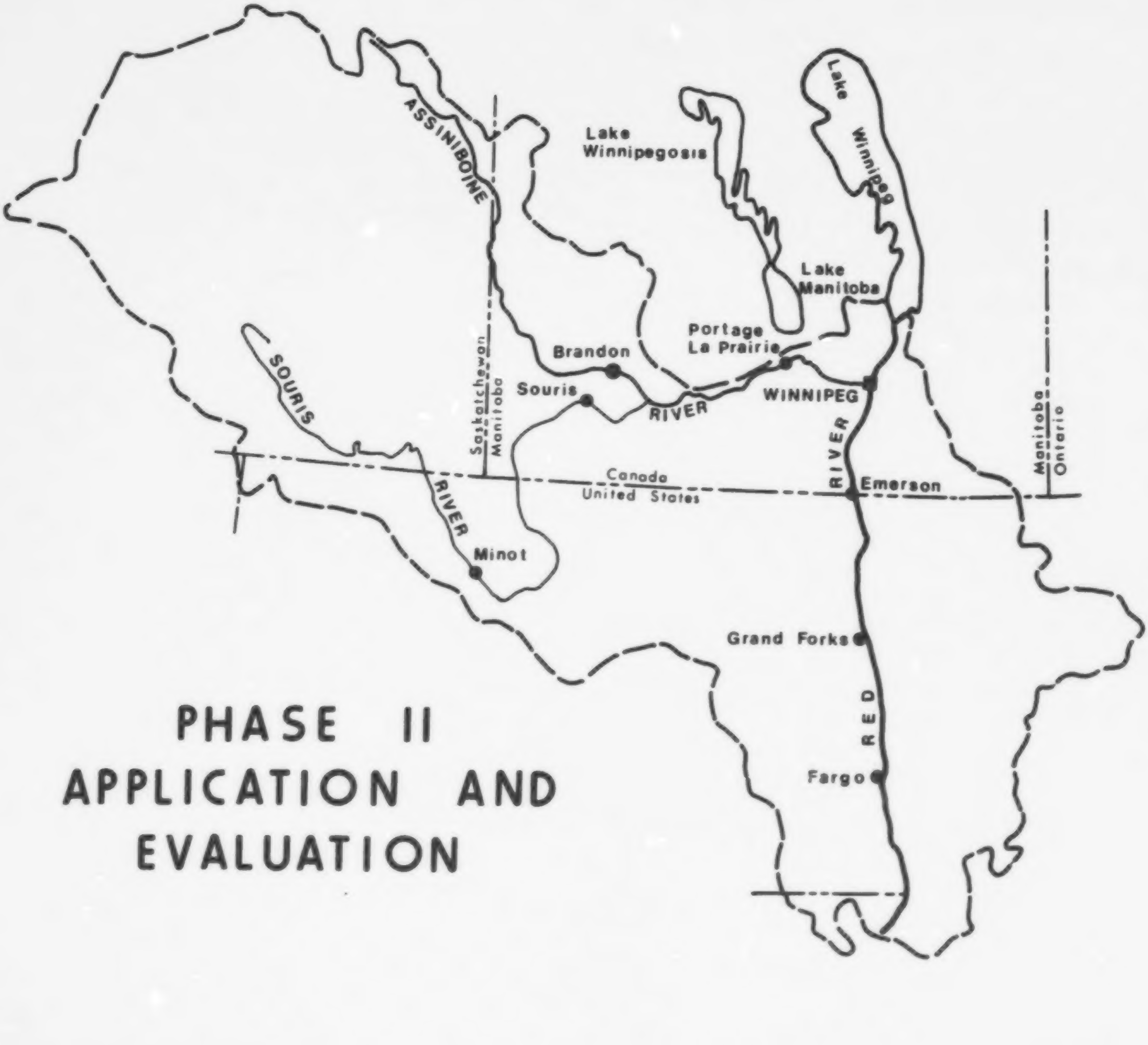
"L. Crilly"
WITNESS

# Volume

# 2

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
AGREEMENT RESPECTING FLOOD FORECASTING

# MANITOBA RIVER FORECAST DEVELOPMENT


ASSINIBOINE
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Lake Winnipeg
Lake Manitoba
Portage La Prairie
Brandon
Souris
SOURIS
RIVER
WINNIPEG
RIVER
Saskatchewan
Manitoba
Manitoba
Ontario
Canada
United States
Emerson
RIVER
Minot
Grand Forks
RED
Fargo

## PHASE II
## APPLICATION AND EVALUATION

Environment Canada

Manitoba Natural Resources

JUNE 1988

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
AGREEMENT RESPECTING FLOOD FORECASTING

# MANITOBA RIVER FORECAST DEVELOPMENT

## PHASE II
## APPLICATION AND EVALUATION

**Environment Canada** **Manitoba Natural Resources**

**Winnipeg, Manitoba**
**June, 1988**

# Synopsis

In March, 1981 the governments of Canada and Manitoba entered into a five-year "Agreement Respecting Flood Forecasting" to examine river flow forecasting methodologies for Manitoba. In the first phase of the study (1981 to 1984) various simulation models were examined and tested on the Boyne River basin in south-central Manitoba. Phase one also involved studies of river channel routing models and tests of real-time data acquisition systems for flood forecasting. The report entitled "Phase One, Planning and Design" documents this work.

The present report entitled "Application and Evaluation" is a sequel to the Phase One report. The report documents the application of the HSPF streamflow simulation model on three watersheds in Manitoba. The report also compares four tributary channel routing models.

The study concluded that the HSPF model does not provide significantly better streamflow simulations than the MANAPI index model which is currently used for flow forecasting in Manitoba. The report recommends that an index type of model of the form of the MANAPI model continue to be used for flow forecasting in Manitoba.

# Résumé

En mars 1981, les governements du Canada et du Manitoba ont conclu une entente de cinq ans concernant la prévision des crues afin d'examiner les méthodes de prévision du débit des riviéres au Manitoba. Dans la première phase de l'étude (1981 à 1984), divers modèles de simulation ont été examinés et essayés sur le bassin de la rivière Boyne dans le centre-sud du Manitoba. La première phase comportait aussi des études portant sur des modèles d'acheminement par les chenaux de rivière ainsi que des essais de dispositifs de saisie des données en temps réel pour la prévision des crues. Le rapport inititulé "Phase One, Planning and Design" documente ces travaux.

Le présent raport intitulé "Application and Evaluation" est une suite du rapport de la première phase. Le rapport documente l'application du modèle de simulation des débits HSPF à trois bassins-versants du Manitoba. On y trouve aussi une comparaison entre quatre modèles d'acheminement par les chenaux des affluents.

L'étude aboutit à la conclusions que le modèle HSPF ne fournit pas de simulations du débit sensiblement meilleures que le modèle à indice MANAPI qui est présentement utilisé pour la prévision des débits au Manitoba. Le rapport recommande qu'un modèle de type à indice de la forme du modèle MANAPI continue à être employé pour la prévision des débits au Manitoba.

# Acknowledgements

This study was conducted under the direction of the Technical Subcommittee on Flood Forecasting. Committee members were:

Environment Canada:
D. R. Kimmett replaced by R. A. Hale (August, 1985)(Co-Chairman)
R. L. Raddatz
J. G. Way (Co-Secretary)

Manitoba Natural Resources:
V. M. Austford (Co-Chairman)
R. J. Bowering
A. A. Warkentin (Co-Secretary)

The project leader was A. A. Warkentin of the Manitoba Water Resources Branch. The simulation studies were performed by G. E. Mohr and E-L. Blais. Preparation of the meteorologic data files was provided by D. N. Walker of the Manitoba Water Resources Branch. The draft report was written by E-L. Blais, G. E. Mohr and A. A. Warkentin. The Technical Subcommittee reviewed the various drafts and made revisions as necessary. The report was typed by E. Boxshall of the Manitoba Water Resources Branch. Drafting support was provided by J. Mamott.

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# Chapter 1

# INTRODUCTION

In March, 1981 the governments of Canada and Manitoba entered into a five-year "Agreement Respecting Flood Forecasting" to examine river flow forecasting methodologies for Manitoba. Phase One (1981-1984) of this Agreement consisted primarily of a Pilot Project, during which various physically based runoff models were applied to the Boyne River basin. Simulations of spring streamflow for selected years were compared to the flows predicted by the Manitoba Antecedent Precipitation Index Model (MANAPI) (Warkentin,1986) which has been the forecasting procedure employed by Manitoba through the 1970's. Phase One also included studies of real-time data acquisition systems and of channel flow routing procedures. A draft report for Phase One has been prepared (Canada-Manitoba Flood Damage Reduction Program; Agreement Respecting Flood Forecasting, Phase One, Planning and Design, 1985). Based on the results of the pilot project the Hydrologic Simulation Program-Fortran model (HSPF) was selected for further study.

During the two year period from April, 1984 to March, 1986, the HSPF streamflow simulation model was applied to the Willow Creek, Gopher Creek and Elgin Creek watersheds in southwestern Manitoba. It was anticipated that better forecasts on these tributaries could be used in conjunction with MANAPI to achieve an overall improvement in flow forecasts for the Assiniboine and Souris Rivers. Calibration of the model on these watersheds presented some difficulties not encountered on the Boyne River. Consequently, some model changes were required and some new approaches to model calibration were implemented. Applications of the HSPF model on the three watersheds are described in Chapter 2.

Studies on tributary channel routing procedures, initiated in Phase One, were completed and are described in Chapter 3 of this report.

Appendix A contains a more detailed discussion of the problems encountered in applying HSPF to a prairie environment and some of the solutions employed. The material it contains may be useful to those intending to apply physically based models in a similar environment.

A technical discussion of flood routing, in general, is contained in Appendix B.

# Chapter 2

# HSPF MODELLING

## 2.1 General Approach

Figure 1 shows the three small watersheds in southwestern Manitoba: Willow Creek, Gopher Creek and Elgin Creek, on which the hydrologic modelling was performed. The application of the HSPF model to these basins illustrated weaknesses in some aspects of the methodology developed during modelling on the Boyne River. As well, deficiencies in how the model simulated certain hydrologic processes were identified. As such, changes were made to improve the methodologies and to improve the model. While some of these changes were required to accommodate different watershed characteristics others were simply the result of the natural evolution of techniques due to the increasing experience of the modellers. Therefore, the techniques applied in the calibration of each of the three basins differed somewhat.

This section contains a discussion of methodologies common to the three basins and a review of the different modelling approaches.

The modelling process was divided into distinct phases. The first of these was the "conceptualization" of each basin's characteristics. Conceptualization involved the determination of the measurable physical characteristics of the basin, the interpretation of non-measurable properties and the configuration of the model in a way that would be representative of the basin. The next step involved the development of an appropriate meteorological data base. The simulation of summer streamflow was then undertaken to calibrate the hydrologic parameters of the model. The simulation of spring conditions followed.

### 2.1.1 Basin Conceptualization

Physical model parameters, such as land surface slope, length of the overland flow plane and effective basin area are measurable basin characteristics which were determined before the start of simulation. The basin area contributing runoff is one of the most important parameters controlling the volume of runoff. The contributing basin area will vary depending upon the magnitude of the runoff event and may equal the gross drainage area for the largest events. The effective drainage area, as defined by the Prairie Farm Rehabilitation Administration (PFRA), is "that portion of a drainage basin which might be expected to entirely contribute runoff to the main stream during a flood with a return period of two years" (PFRA, 1983). The noneffective portion is considered to contribute only marginally in all but the wettest periods. During modelling it appeared that most events could be simulated with reasonable accuracy using PFRA's definition of effective drainage area.

Gross and effective drainage areas were determined using Manitoba Water Resources watershed maps, 1:50,000 topographical maps, air photographs and PFRA drainage area maps. As the available topographical maps had a 25 foot contour interval some judgement was required in determining effective drainage areas.

Channel routing parameters were determined from the estimated physical characteristics of the streams. The HSPF routing subroutine uses a series of connected reservoirs to represent the stream channel. Storage versus outflow relationships are required for each reservoir. The methods used to derive these routing parameters varied from basin to basin depending upon the quality and availability of data concerning stream channel characteristics. A lack of cross-section data for the streams under study necessitated the use of simplified stream channel concepts which did not require survey data to define the routing relationships. The channels were assumed to have triangular cross-sections (see Appendix A, Section 3.4.1). The number of "reaches" required to adequately describe the stream channel varied from basin to basin depending on the complexity of its channel network. The land surface contributing to each channel section was determined in conjunction with the routing conceptualization.

Although topography and soils varied slightly over each basin, they were relatively homogeneous over their effective drainage areas. Also local variations in precipitation were unknown due to the sparseness of the meteorological network. Therefore, each watershed was conceptualized as a single uniform segment for hydrologic modelling.

### 2.1.2 Meteorological Data Base

A thorough examination of the Atmospheric Environment Service meteorological data archives, performed at the beginning of the study, identified the climatological and mainline[1] meteorological stations in and around the study basins. Basin daily precipitation amounts were computed using the available climatological station data and Thiessen Polygon analysis (Thiessen, 1911). The hourly rainfall distribution, as recorded at Brandon Airport, (the closest mainline station for all three basins) was used to divide the daily basin values into hourly values.

---

[1] Hourly and synoptic observations

Some of the meteorological data files were common for all three basins. Daily pan evaporation was estimated by averaging pan evaporation measured at Baldur, Manitoba and Broadview, Saskatchewan. Wind travel measured at Brandon was used for all basins. Solar radiation was computed by relating bright sunshine data from Brandon to solar radiation measured at the Winnipeg Airport. This method of estimating solar radiation was developed during Phase One (Agreement Respecting Flood Forecasting, Phase One, Planning and Design, 1985). Hourly air temperatures and dew point temperatures recorded at Brandon were used for Elgin Creek and Willow Creek modelling. For Gopher Creek modelling air temperatures and dew point temperatures at Brandon and Broadview were averaged. The meteorological network is shown on Figure 2.

Meteorological data files were produced for synoptic time periods (four data values per day) for the spring period. The six-hour time interval was selected as a compromise between daily and hourly intervals. The daily interval was considered too long to adequately represent temporal meteorological changes, particularly with respect to snowmelt calculations. The hourly interval was considered unnecessarily short. Also, precipitation data would only be available at six hour intervals during operational use of the model. Hourly data files were however, prepared for the summer period where high rainfall intensities necessitated their use in simulating infiltration. Data time series were not developed for the winter since there is typically no winter snowmelt runoff in Manitoba. Winter precipitation was obtained from Thiessen Polygon analyses (1911) or by simple averaging of cumulative winter precipitation at meteorological stations.

### 2.1.3 Summer Modelling

The years selected for summer simulations were those where the fall soil moistures were required as starting conditions for simulation the following spring. As well, some years were selected where streamflow was significant enough to aid in the calibration of the model.

Early in the modelling process, it was decided to calibrate the model parameters which define overland and subsurface flow (lands phase) during the simulation of summer conditions, when watershed response could be studied for a variety of precipitation inputs and soil moisture conditions without the extra uncertainties of snowmelt inputs and possible frozen soils. It was assumed that the calibration of these parameters would be possible if there were a sufficient number of runoff events and reasonably accurate precipitation data. Experience showed that the intense precipitation rates associated with summer thunderstorms required the HSPF model to be run on an hourly time step to achieve reasonably accurate simulations.

Summer simulations covered the period from May 1 to October 31. This corresponds to the period for which pan evaporation observations were available.

The methods used to calibrate the model are discussed in Appendix A. Initially the same methods were applied for modelling both the Willow and Gopher Creek basins. The soil profile was conceptualized as an upper and lower zone. Initial moisture storages in upper and lower zones were based on spring runoff conditions and streamflow prior to the start of simulation. Groundwater and interflow storages were estimated using hydrograph separation. The lands phase parameters employed were those suggested in the various HSPF model users manuals or the parameters found to be successful when modelling segments of the Boyne River watershed during Phase One. Calibration involved multiple runs and the adjustment of parameters to achieve an acceptable overall simulation.

One of the major problems encountered was that the specification of the initial lower zone soil moisture affected the entire summer run. Inaccuracies in estimating the starting values, therefore, hindered model calibration. Despite this problem, acceptable summer simulations were achieved on Gopher and Willow Creeks.

The results of the summer runs were evaluated by visual comparison of observed and simulated hydrographs. Examining hydrographs proved to be a better indicator of model performance than comparing simulated and observed runoff volumes for the season. This was due to the fact that the total summer runoff volume for the three basins over the period of record averaged less than eight millimetres per year, which is less than the potential error in the estimated summer precipitation data and in the model's calculation of evapotranspiration. The error in precipitation estimates for the growing season at ungauged points in Canada's eastern Prairies has been estimated at 19 percent (Raddatz and Kern, 1984). If this percentage were applied to the normal precipitation in southwestern Manitoba for the period from May 1 to October 31, the absolute error would be approximately 63 millimetres. Although the error associated with the area-average precipitation estimate for a watershed would be somewhat less than for a point estimate, it would likely still be several times larger than the surface runoff from these basins for the same period.

The evapotranspiration component of the model was not specifically calibrated since reliable measures of this parameter were unavailable. From the discussion in the preceding paragraph it can be seen that it would have been difficult, if not impossible, to calibrate evapotranspiration indirectly as a residual of the water balance.

Hydrograph plots illustrated inconsistencies between estimated basin rainfall data and streamflow data. In some cases these plots also gave an indication of whether the computed area-averaged rainfall intensities were correct. An increase in observed streamflow was considered a good indicator of the occurrence of a precipitation event. A significant increase in streamflow that did not have a corresponding rainfall event was therefore an indication that the computed precipitation was in error unless releases from upstream storages had occurred. Similarly, the lack of an observed streamflow event associated with an apparent significant rainfall event also suggested an error in the data. Errors in rainfall intensity were suspected if the shapes of the simulated and observed hydrographs were dramatically different even though the daily rainfall volumes appeared correct.

The difficulties experienced in calibrating Elgin Creek for the summer period led to a reassessment of whether visually examining simulated hydrographs was the only way to check simulations. Comparison of simulated end of summer lower zone soil moisture against observed soil moisture at the nearest Manitoba Water Resources fall soil moisture site was introduced as a second check on simulation accuracy. Using the upper 430 millimetres of the soil profile the field soil moisture data were converted to a depth of water assessed to be roughly equivalent to lower zone storage, by the method outlined in Section 2.2.1 of Appendix A.

Willow and Elgin Creek simulations were then recalibrated using observed fall soil moistures as well as hydrograph shape to check the calibrations. Gopher Creek summer simulations were not rerun since it was felt that recalibrating the model to achieve soil moistures which were closer to field observations would not significantly improve the simulation of streamflows.

During summer modelling on all basins it was noted that the basin's hydrologic response varied throughout the summer in conjunction with changes in plant growth and tillage practices. There is provision in the model to account for these changes. Some parameters can be specified on a monthly basis. The model performs a linear interpolation between the monthly values to ensure that there are no discontinuities in the parameters. Parameter values were input to the model for the first day of each month and were estimated on the basis of normal plant growth stage. In the final summer calibration runs, slight adjustments were made to the monthly upper zone nominal storage and the evapotranspiration parameters to better account for the changes in hydrologic response caused by plant growth.

The model also allows variables to be updated in the midst of a simulation run. This feature (called "SPEC-ACTION") allows a parameter value to be changed on a specific date. This was used to specify the average dates when planting and harvesting were generally completed in southwestern Manitoba (Manitoba Department of Agriculture, 1983). Parameters affected by these activities (e.g. storage in the upper soil zone) were changed on these dates to better reflect hydrologic response. Since these activities tended to occur during periods of low plant growth and during dry weather the use of this feature did not appear to cause any discontinuities in the simulations.

### 2.1.4 Spring Modelling

The melt parameters derived for the Willow Creek Watershed, the first basin studied, were also used for modelling Elgin and Gopher Creeks. Snowmelt simulations were run using data from March 1 to either the end of April or May depending on the date of completion of snowmelt. The model was run on a six hour time step.

Infiltration is one of the key processes affecting runoff and is substantially different for frozen and unfrozen soils. Numerous observations in Saskatchewan (Granger et. al., 1984) have shown that meltwater was confined to an average depth of 26 cm with a standard deviation of 10 cm. The shallow depths can be attributed to refreezing of meltwater in the frozen soil thereby causing a blockage of flow. To model this process meltwater must be restricted from entering the lower soil profiles and be kept at or near the surface. The HSPF model does not contain provisions for modelling changing soil frost conditions. However, it was possible to approximate this process by one of two methods. For Willow and Elgin Creeks the spring infiltration rate was set to its minimum allowable value, while for Gopher Creek the Ice Flag option was employed along with a slightly reduced, summer infiltration rate. The two procedures achieved the same division between interflow and overland flow. A more detailed discussion can be found in Section 3.2 of Appendix A.

Starting soil moistures used were those computed near freeze-up from the previous summer runs. That is, it was assumed that soil moistures at the start of spring simulation would be the same as that at freeze-up of the previous year. This was thought to be a reasonable assumption since the soil moisture would tend to be locked into the soil column until the spring thaw.

Pan evaporation data was not available during the early spring. Evaporation was assumed to be zero. This assumption was not considered to be a problem since evaporation losses should be small during the relatively short runoff period, especially considering the cold water temperatures.

## 2.2 The Willow Creek Watershed

### 2.2.1 Basin Description

#### 2.2.1.1 Location

The Willow Creek basin is located just northeast of the City of Brandon in southwestern Manitoba as shown on Figure 1. The headwaters of the basin are located near the junction of Provincial Trunk Highway (P.T.H.) 24 and 10. Willow Creek empties into the Assiniboine River just downstream of Brandon.

#### 2.2.1.2 Basin Characteristics

The basin has a gross drainage area of 570 square kilometres (220 square miles). The effective drainage area was determined to be 400 square kilometres (154 square miles) which is 70 percent of the gross basin area. As shown on Figure 3 there are two major noneffective areas, an area of 65 square kilometres (25 square miles) located in the northwestern portion of the watershed and an area of 105 square kilometres (40.5 square miles) located in the eastern portion of the watershed. The former area is typical pothole country formed by stagnant glacial ice and has no well defined drainage network. The latter area, South Boggy Creek and its tributaries, drains into Douglas Marsh. The Marsh is composed of deltaic deposits and has no well defined outlet to Willow Creek.

The basin is bounded on the north and west by the Little Saskatchewan River, and on the south by the Assiniboine River. The eastern boundary is erratic but generally follows the "fall line" of the Riding Mountain escarpment. The southeastern portion balloons out into an area known as Douglas Marsh where the drainage divide is not well defined.

The basin has many well defined north-south feeder streams that have become incised over millennia as water flowed off the southern slope of Riding Mountain. These feeder streams collect in a main channel near the bottom of the basin just before the river drops to the Assiniboine River.

As shown in Figure 4, the elevation of the basin ranges from 550 metres (1800 feet) in the headwaters to 365 metres (1200 feet) at the streamflow gauge near the river mouth. The basin has an average land surface slope of 0.0052. This slope is fairly uniform across the basin except for the Douglas Marsh area.

#### 2.2.1.3 Soils

The soils of the Willow Creek basin are the product of a glacial environment. The basin has two main geological features, a till plain and a swampy area of deltaic deposits as shown on Figure 4. The till deposits are a mixture of gravel, sands, silts and clays laid down by the melting glacier. In the southeastern portion of the Willow Creek basin, Douglas Marsh is situated on the western extremity of the Assiniboine River delta formed during glacial times. Geological features of lesser areal extent are the pockets of lacustrine clays deposited when the basin was partially covered by glacial Lake Souris.

#### 2.2.1.4 Hydrometeorological Network

Figure 2 shows the location of the meteorological stations used in the study.

Precipitation data was recorded at four stations; Brandon Airport (A.), Brandon CDA, Minnedosa and Neepawa. Both Brandon stations have hourly precipitation records and Nipher-shielded snow gauges. The Thiessen Polygon weights used to compute basin area-average daily precipitation for the summer period were: Brandon A.-34 percent, Brandon CDA-33 percent, Minnedosa-17 percent and Neepawa-16 percent.

Streamflow records were obtained from the Water Survey of Canada station; Willow Creek near Chater (05MH011). Only daily flow records were available for the years modelled.

Fall soil moistures were obtained from the Province of Manitoba soil survey site A-IV-1 located on the southern edge of the basin.

### 2.2.2 Modelling Under Summer Conditions

The HSPF model was calibrated using the years 1968, 1969, 1970, 1971, 1973, 1974, 1975 and 1978.

The HSPF's land segment was run and the outflow was passed to the channel routing portion of the model. The basin was divided into six routing segments as shown on Figure 3. The routing configuration involved combining numerous small feeder channels into two channels labelled the west channel and the main channel.

Each channel section was conceptualized as having a side slope that got progressively flatter moving upstream as was observed from the limited cross-section data available along the main stem. Routing tables were computed using the channel slope, computed cross-sectional area and Manning's formula (Manning, 1891).

#### 2.2.2.1 Calibration

Those parameters which had the largest effect on basin runoff, termed major parameters, were adjusted first. Those parameters which had a localized or minimal effect on the overall calibration, termed minor parameters, were adjusted later.

The major group of inputs and parameters adjusted during calibration were those pertaining to effective drainage area, depressional storage, infiltration rate, dead storage in channels, nominal lower zone storage, lower zone evapotranspiration rate and initial storage levels in the upper and lower soil zones. Six runs were made to calibrate this group of parameters.

The minor group of inputs and parameters adjusted during calibration were those pertaining to seasonal depressional storage, stage of plant growth and interception storage. In some cases area-average precipitation estimates were also adjusted. After each run it was determined whether or not parameter adjustment had made an overall improvement to the simulations. Nine runs were made to calibrate this group of parameters.

Other HSPF parameters not discussed above were set to default values as described in Appendix A.

#### 2.2.2.2 Results

Willow Creek's summer model runs were evaluated by visually examining observed and simulated hydrographs and by comparing measured fall soil moistures against simulated fall lower zone soil moistures. A good correlation between simulated and observed fall soil moistures served as a check of the model's calibration.

Inspection of summer hydrographs showed that HSPF simulated summer rainfall events with reasonable accuracy. Simulated fall lower zone soil moisture was generally in fair agreement with fall soil moisture measured at a specified point in the province's soil moisture survey as shown in Table 2.1. In 1974 and 1978 however, HSPF's generated soil moisture was considerably lower than the observed soil moisture. The low HSPF value was attributed to the internal HSPF drying parameter. This parameter, which affects the rate of evapotranspiration, may have been set too high for soils in the watershed. On the other hand, the point fall soil moisture survey data may not be representative of the watershed.

Runoff volumes, which are also listed in Table 2.1, were not compared for reasons discussed in Section 2.1.

### 2.2.3 Modelling Under Spring Conditions

Data for five median to high flow years were used to calibrate the model for spring runoff. The years were 1969, 1970, 1974, 1976 and 1979.

The model configuration was similar to that for summer runs with one land segment and six routing segments.

Starting conditions for soil moisture were obtained from the previous summer run. It was assumed that soil moistures at the start of spring simulation would be the same as on the previous October 31.

Estimates of winter snowpack were made by averaging the accumulated snowfall from Nipher-shielded gauges near the basin; namely, Brandon Airport and Brandon CDA. Although data from a number of climate stations were available, only those for Nipher gauges were used because it is generally believed that the Nipher-shielded gauges provide better estimates of winter precipitation than ordinary climatological stations.

#### 2.2.3.1 Calibration

The initial spring simulation runs on Willow Creek greatly underestimated observed runoff values. Assuming the effective drainage area remains relatively constant from the summer to the spring, one method of simulating more runoff would be to reduce the meltwater storage capacity of the soil. To restrict the downward movement of water the infiltration parameter was reduced from 1.27 mm per hour to a minimum value of 0.0025 mm per hour and the upper zone was used as a confinement zone for meltwater.

After a series of trial runs it was determined that an upper zone nominal storage (UZSN) of 33 millimetres (1.3 inches), the same as that used for the summer runs, gave the best estimate of spring runoff volume for all years except 1979. In 1979, the simulated runoff was double the observed. It seemed unlikely that such a large error

Table 2.1: Willow Creek Summer Simulation Results[a]

| Year | May Soil Moisture (mm) | | | | Summer Precip. (mm) | Evapotranspiration[e] (mm) | Runoff (mm) | | Oct. 31 Soil Moisture (mm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | UZS[b] | Fall SMI[c] | + Spring Recharge | = LZS[d] | | | Simulated | Observed | UZS[b] | LZS[d] | SMI[c] | LZS[d]/SMI[c] (%) |
| 1968 | 13 | 132 | 0 | 132 | 376 | 302 | 10 | 3 | 6 | 179 | 183 | 98 |
| 1969 | 20 | 183 | 25 | 208 | 320 | 338 | 33 | 23 | 7 | 160 | 152 | 105 |
| 1970 | 28 | 152 | 38 | 190 | 335 | 358 | 28 | 25 | 8 | 158 | 150 | 105 |
| 1971 | 13 | 150 | 13 | 163 | 460 | 368 | 51 | 30 | 28 | 172 | 175 | 98 |
| 1973 | 0 | 102 | 0 | 102 | 419 | 315 | 5 | 3 | 14 | 168 | 180 | 93 |
| 1974 | 33 | 180 | 25 | 205 | 206 | 323 | 36 | 25 | 2 | 98 | 127 | 77 |
| 1975 | 33 | 127 | 25 | 152 | 429 | 399 | 20 | 15 | 14 | 168 | 178 | 94 |
| 1978 | 20 | 142 | 13 | 155 | 254 | 318 | 5 | 5 | 1 | 95 | 142 | 66 |

Notes:

[a] Results from WILSUM15.UCI; with infiltration of 1.27 mm/hr, lower zone nominal storage (lzsn) of 203 mm, maximum monthly lower zone evapotranspiration (lzetpm) of 0.54 and monthly upper zone nominal storage (uzsnm) ranging from 20.6 to 33.0 mm.

[b] UZS is the upper zone storage.

[c] SMI is the soil moisture index. It is the depth of water measured in the top 430 mm of the soil.

[d] LSZ is the lower zone storage.

[e] Combination of plant interception, upper zone evapotranspiration and lower zone evapotranspiration.

would result from an error in the overwinter precipitation amounts. The most probable reason was that in 1979 more water had been absorbed by the soil than in the other years. Gray has reported that under low soil moisture conditions soil cracking occurs in clay based soils which results in increased water infiltration into the soil. Fall soil moistures in 1978 were low. An attempt was made to simulate the effect of soil cracking by using the same infiltration rate for the lower zone as that used during summer simulations. This however, resulted in an underestimate of runoff. It appeared, therefore, that in 1979 cracked frozen soil had a meltwater absorption capacity less than the same soil in summer but greater than during normal spring periods. Soil cracking was finally modelled by increasing upper zone nominal storage. A value of 51 millimetres (2.0 inches) for UZSN gave the best results for 1979. For future simulations soil cracking may be suspected whenever the fall simulated conditions show a near zero upper zone storage level and a below normal lower zone storage level.

#### 2.2.3.2 Results

Table 2.2 and Figures 5 to 9 show the results of the spring simulation runs. The table shows that simulated spring runoff volumes ranged from 91 percent to 112 percent of the observed. The results show that restricting the storage capacity to only the upper zone was successful in modelling spring runoff volume; although a modification to the upper zone storage approach was required in 1979 when soil cracking was suspected.

Simulated and observed peak discharges showed a much larger variation than runoff volume. It was found that a small change in the shape of the simulated hydrograph caused a substantial increase or decrease in the peak discharge. Sensitivity tests showed that simulated peak discharges were sensitive to slight changes in melt factors, such as the albedo and the meteorological inputs causing snowmelt and to the total basin precipitation amount.

The timing of the simulated snowmelt peak was generally within one day of the observed daily peak.

#### 2.2.3.3 Model Verification

During calibration of a model one or two years of data are normally excluded from the calibration process so that the ability of the model to simulate runoff using information not included during calibration can be tested. This is referred to as model verification. Unfortunately this could not be done for Willow Creek. Adequate spring runoff data was only available for five years and all years were required during the calibration process to cover the full variety of runoff events that occur in the Basin.

Table 2.2: Willow Creek Spring Summary

| Year | Observed Runoff ( mm ) | Runoff Coef. % | Simulated Runoff ( mm ) | Simulated as % of Observed | Observed Peak ( $m^3$/sec ) | Simulated Peak ( $m^3$/sec ) | Timing |
|---|---|---|---|---|---|---|---|
| 1969 | 75 | 48 | 68 | 91 | 77.9 | 68.7 | 1 day early |
| 1970 | 81 | 49 | 76 | 94 | 17.0<br>58.3 | 33.6<br>40.2 | on time<br>on time |
| 1974 | 82 | 52 | 76 | 93 | 80.4 | 69.9 | 1 day early |
| 1976 | 75 | 48 | 76 | 101 | 56.9 | 84.0 | 1 day early |
| 1979 | 51 | 28 | 57 | 112 | 71.4 | 63.2 | 1 day early |

**Notes:**

- Results from run TEST9SPR.UCI.
- Runoff depths are based on the effective drainage area of the watershed.
- Runoff coefficient is the ratio of observed runoff depth to basin area-average precipitation.
- There were three distinct peak discharges in 1970.

## 2.3 Gopher Creek Watershed

### 2.3.1 Basin Description

#### 2.3.1.1 Location

The location of the Gopher Creek basin, in southwestern Manitoba and southeastern Saskatchewan is shown on Figure 1. It starts near the Town of Moosomin, Saskatchewan and joins the Assiniboine River near the Town of Virden, Manitoba.

#### 2.3.1.2 Basin Characteristics

The drainage basin is 58 kilometres (36 miles) in length and is less than eight kilometres (five miles) in width. The effective drainage area of the Gopher Creek watershed was determined to be 152 square kilometres (58.8 square miles) or only 43.4 percent of the gross drainage area of 350 square kilometres (135.1 square miles). This percentage is fairly typical of the glaciated prairies where melting blocks of ice have left the land surface dotted with shallow depressions. Stream channels contained within the basin total about 150 kilometres (93 miles) in length, if major tributaries are included. This is about one kilometre of channel for every square kilometre of basin. It is likely that some of the areas that have been designated noneffective are closed depressions which will remain noncontributing for all but the largest events.

The Gopher Creek watershed is located on gently rolling deposits of ground moraine which are part of the Oxbow Till Plain. Gopher Creek itself is the longest of several small intermittent streams which drain shallow depressions in the till plain. These small streams tend to flow parallel to each other separated by low ridges and at intervals join Gopher Creek as it flows in a southeasterly direction from its headwaters. Over time, the main channels have become incised into the underlying glacial till and soft shale bedrock whereas the smaller streams simply flow along gentle swales.

South of Virden, the stream emerges onto the Lake Souris lacustrine plain. At this point there is a reduction in gradient and a change in the direction of the land surface slope which causes the creek to flow northward toward the Assiniboine River.

The Gopher Creek basin ranges in elevation from 570 metres (1875 feet) in the headwaters to 440 metres (1450 feet) at the stream flow gauge near P.T.H. No. 83. The average slope of the land surface is 0.0083. Channel slope varies from 0.0028 in the upper reach to as low as 0.0013 through the middle reaches.

#### 2.3.1.3 Soils

The soil of the Gopher Creek basin has been formed on materials deposited from the basal surface of the continental ice sheet. The till has a fairly high clay content so that the soil that has formed is classified texturally as a loam to clay loam. The water holding capacity of this type of soil above the hygroscopic coefficient is typically 21.8 cm/m (2.62 inches/foot) under cropped conditions. This is roughly equivalent to 88 percent of the pore space of the soil.

#### 2.3.1.4 Hydrometeorological Network

Precipitation data was available for six climatological stations in or near the basin; Fleming CDA, Fleming South, Maryfield, Moosomin, Virden and Virden Scarth as shown in Figure 2. None of these stations have Nipher-shielded snow gauges. Thiessen polygon analysis indicated that the daily precipitation amounts recorded at these stations should be evenly weighted to produce the basin area-average daily precipitation.

Daily stream flow was available for the Water Survey of Canada Station Gopher Creek near Virden (05MG003), located at the bridge along the grid road running west from P.T.H. No. 83 approximately 6.5 kilometres (4 miles) south of Virden.

The Manitoba Water Resources soil moisture site closest to the Gopher Creek basin is located 3.2 kilometres (2 miles) south of Virden along P.T.H. No. 83.

### 2.3.2 Modelling Under Summer Conditions

The model was calibrated for Gopher Creek using summer data for the years 1968, 1969, 1970, 1971, 1973, 1975, and 1978.

As shown in Figure 10, the basin was subdivided into five segments for summer simulations. This was done solely to facilitate the routing of simulated flows in a manner that would be representative of the basin's shape. Meteorological inputs and model parameters were identical for each segment. The main channel network within the Gopher Creek basin consists of one main southern channel and Gopher Creek which flows along the northern side of the basin. The northern channel has numerous small tributaries. In order to simplify the channel network it was conceptualized as one channel divided into five reaches. The effective contributing area for each of these five reaches was determined from a 1:50,000 topographic map.

Routing parameters were determined using the topographic map method described in Appendix A. The storage volume for a given depth of flow was the combined storage of all channels within the reach. Stage versus discharge tables for the end of reach locations were derived by combining the individual rating tables for each channel leaving the reach. These rating tables were computed using Manning's formula (Manning, 1891). The end points of reaches were chosen at locations in the basin where the channel network became simplified, hence errors resulting from combining rating curves should be minimized.

#### 2.3.2.1 Calibration

The calibration of the HSPF model on Gopher Creek was largely a trial and error process. The only parameters which were not subject to this process were those representing the basin's physiographic features of drainage area, land surface slope, length of overland flow path, and channel characteristics. These were determined before simulation began. The remaining parameters were initially set to either the values determined from previous HSPF modelling or the suggested values from the various reports (Donigian et. al., 1978 and CH2M Hill, 1980). During subsequent runs the model parameters were adjusted to improve the overall simulation.

The first parameter to be adjusted was the one which controlled the rate of infiltration. This parameter was adjusted until the simulated peaks were of the same order of magnitude as those observed. During calibration, emphasis was placed on the accurate simulation of peak discharges rather than runoff volumes because of the small volume of runoff from summer events as described in Section 2.1. Further adjustment of the infiltration parameter was not made until the nominal storages had been examined and adjusted.

Small runoff events were examined and used to estimate the parameter which represents an index value of the water stored in the top few centimeters of soil (upper zone nominal storage). This approach was taken on the premise that after a dry period the volume of water held in this zone would be close to zero. If rainfall resulted in a small amount of runoff under this condition it would have had to just exceed the upper zone nominal storage. The nominal storage parameter was, therefore, set so that the model would produce a small response from such storms. This gave a reasonable initial estimate of nominal storage for fine tuning during subsequent runs. In the final version of the summer model, slight adjustments were made to nominal storage on a monthly basis to better account for the changes caused by agricultural practices and plant growth.

The model calibration of the hydrologic response in the Gopher Creek watershed was less sensitive to other parameters. Calibration for these parameters became somewhat intuitive as adjustments were made by trial and error to improve the overall simulation.

#### 2.3.2.2 Results

The results of summer modelling on Gopher Creek were relatively good, based on a visual examination of the hydrographs. This success was attributed to the relative simplicity of the basin in terms of soils, topography, and to the channel network. In the Gopher Creek watershed, the routing parameters could be determined from topographic map information. This method seemed to produce parameters which adequately reflected the true nature of the stream.

Despite the somewhat promising overall results, runoff from many of the summer storms was simulated very poorly. This was attributed to unrepresentativeness of the basin area-average precipitation particularly for localized summer storms. The results of summer modelling are shown in Table 2.3.

### 2.3.3 Modelling Under Spring Conditions

The model was calibrated using data for the years 1969, 1970, and 1974. The conceptualization of the basin was not significantly altered from that for the summer simulations.

Accumulated winter, precipitation (November 1 to February 28) was estimated using data from the six climatological stations listed on Section 2.3.1. These values were weighted using Thiessen Polygon analysis to produce basin area-average winter precipitation totals for use as starting values for spring simulations.

Table 2.3: Gopher Creek Summer Simulation Results$^a$

| Year | May Soil Moisture (mm) |  | Summer Precip. (mm) | Evapotranspiration$^d$ (mm) | Runoff (mm) |  | Oct. 31 Soil Moisture (mm) |  |
|---|---|---|---|---|---|---|---|---|
|  | UZS$^b$ | LZS$^c$ |  |  | Simulalted | Observed | UZS$^b$ | LZS$^c$ |
| 1968 | 25 | 191 | 335 | 414 | 10 | 2 | 13 | 106 |
| 1969 | 29 | 203 | 292 | 411 | 17 | 17 | 14 | 80 |
| 1970 | 51 | 241 | 399 | 526 | 28 | 39 | 23 | 103 |
| 1971 | 22 | 241 | 353 | 450 | 11 | 24 | 29 | 116 |
| 1973 | 0 | 191 | 401 | 450 | 6 | 7 | 19 | 110 |
| 1975 | 51 | 241 | 455 | 531 | 44 | 35 | 30 | 146 |
| 1978 | 5 | 165 | 325 | 414 | 3 | 2 | 3 | 66 |

Notes:

[a] Results from GOPHSUM.UCI; with infiltration of 0.76 mm/hr, lower zone nominal storage (lzsn) of 190.5 mm, maximum monthly lower zone evapotranspiration (lzetpm) ranging from 0.0 to 0.72 and monthly upper zone nominal storage (uzsnm) ranging from 25.4 to 38.3 mm.

[b] UZS is the upper zone storage.

[c] LSZ is the lower zone storage.

[d] Combination of plant interception, upper zone evapotranspiration and lower zone evapotranspiration.

#### 2.3.3.1 Calibration

Various infiltration reduction scenarios and methods of estimating initial storage values for the upper and lower soil zones were tested during trial simulations. It was determined that the Ice Flag utility of the model and a reduced infiltration rate from 0.76 mm per hour to 0.25 mm per hour could be used to approximate the infiltration changes caused by ice in the soil for the period up to the runoff peak.

An improved method of estimating starting soil moistures was determined over several trial runs. The assumption was made that the index of storage capacity within the lower soil zone (lower zone nominal storage) was related to the porosity of the soil and that 100 percent saturation of the soil would occur at the nominal value. The field capacity of soil in the Gopher Creek watershed was found to be 88 percent of saturation. Even if the soil were saturated in the fall it could be expected to decrease to field capacity by the spring. Therefore, 16.8 centimetres (6.6 inches), which is 88 percent of the lower zone's nominal storage, was set as the maximum volume of lower zone storage (LZS) at the start of spring runoff. The percent of saturation measured during the fall soil moisture survey at the Virden station was multiplied by 19 centimetres (7.5 inches), the value of lower zone nominal storage, to obtain the starting spring value of lower zone storage.

The possibility of a cold snap occurring soon after a rainfall event in late fall which might "lock-in" the soil moisture above field capacity, was ignored. This seemed to be a reasonable simplification considering the time constraints of this study. The simulations did not appear to be adversely affected by this assumption.

The starting spring upper soil zone storage was set at the value computed for October 31 of the previous year's summer simulation. The value of upper zone nominal storage for spring calibration was the same as that determined from summer calibration.

#### 2.3.3.2 Results

Simulated runoff volumes were less than observed volumes in all three of the calibration years, but peak flows were higher than observed in 1969 and 1974. The results of calibration runs are shown in Table 2.4 and on Figures 11 to 13.

#### 2.3.3.3 Verification

In order to test the model's calibration and the methodologies developed for dealing with estimating initial soil moistures, verification runs were performed using input data for 1976 and 1979. The results are shown in Table 2.5 and on Figures 14 and 15.

Table 2.4: Gopher Creek Spring Calibration Summary

| Year | Observed Runoff ( mm ) | Runoff Coef. % | Simulated Runoff ( mm ) | Simulated as % of Observed | Observed Peak ( $m^3/sec$ ) | Simulated Peak ( $m^3/sec$ ) | Timing |
|---|---|---|---|---|---|---|---|
| 1969 | 65 | 39 | 51 | 78 | 17.3 | 19.4 | 1 day early |
| 1970 | 60 | 36 | 44 | 73 | 8.6<br>6.5 | 3.6<br>6.5 | on time<br>1 day early |
| 1974 | 61 | 36 | 56 | 92 | 13.4 | 19.2 | on time |

**Notes:**

- Results from GOPHERSPR2A.UCI
- Runoff depths are based on the effective drainage area of the watershed.
- Runoff coefficient is the ratio of observed runoff depth to basin area-average precipitation.
- There were two distinct peak discharges in 1970 separated by heavy additional snow.

Table 2.5: Gopher Creek Spring Verification Summary

| Year | Observed Runoff ( mm ) | Runoff Coef. % | Simulated Runoff ( mm ) | Simulated as % of Observed | Observed Peak ( $m^3/sec$ ) | Simulated Peak ( $m^3/sec$ ) | Timing |
|---|---|---|---|---|---|---|---|
| 1976 | 169 | 82 | 104 | 61 | 46.2 | 36.7 | 1 day early |
| 1979 | 40 | 21 | 48 | 118 | 10.8 | 13.8 | on time |

**Notes:**

- Runoff depths are based on the effective drainage area of the watershed.
- Runoff coefficient is the ratio of observed runoff depth to basin area-average precipitation.

## 2.4 The Elgin Creek Watershed

### 2.4.1 Basin Description

#### 2.4.1.1 Location

The Elgin Creek basin is located in southwestern Manitoba as shown on Figure 1. The headwaters of the basin are located on the north slope of the Turtle Mountain escarpment. The stream empties into the Souris River just downstream of Souris, Manitoba.

#### 2.4.1.2 Basin Characteristics

The Elgin Creek watershed has a gross drainage area of 1187 square kilometres (458 square miles). The effective drainage area, as shown on Figure 16, was determined to be considerably smaller at only 193 square kilometres (74 square miles) or 16 percent of the gross drainage area. The effective area does not include Whitewater Lake or its tributaries since this portion of the watershed has not overflowed into Elgin Creek in recent history. This has been confirmed by air photos of Whitewater Lake taken in the spring of 1976 when Elgin Creek flows were the highest on record.

A small water supply reservoir with a storage of 615 cubic decametres (500 acre-feet) at full supply level (fsl) is located near the town of Elgin. As the available storage capacity in the spring is usually 60 to 125 cubic decametres (49 to 101 acre-feet) the reservoir has only a negligible effect on the attenuation of spring flows.

The basin has many well defined north/south feeder streams north of Whitewater Lake which merge into two main channels in the middle of the basin and finally into one channel shortly before the Creek enters the Souris River. The river channels are more incised in the middle and lower portions of the basin.

The elevation of the basin ranges from 495 metres (1625 feet) in the headwaters to 425 metres (1400 feet) at the streamflow gauge near the river mouth. The basin has an average land surface slope of 0.0027 and an average stream channel slope of 0.0013.

#### 2.4.1.3 Soils

The soils of the Elgin Creek basin are a product of a glacial environment. In the middle and upper portions of the basin the soils are composed of glacial till, a mixture of gravel, sands, silts and clays. Soils in the lower portion of the basin are composed of lacustrine deposits composed mostly of silts and clays from glacial Lake Souris. Both glacial deposits are relatively impervious.

#### 2.4.1.4 Hydrometeorological Network

Figure 2 shows the location of the meteorological stations.

Daily precipitation data was available from three climate stations in or near the basin; Boissevain, Deloraine and Caroll. Thiessen Polygon weights used to compute summer basin precipitation were: Caroll-37 percent, Deloraine-25 percent and Boissevain-38 percent.

Streamflow records were obtained from the Water Survey of Canada station; Elgin Creek near Souris (O5NG012). Only daily flow records were available for the years used in modelling.

Fall soil moistures were obtained from measurements at Manitoba Water Resources soil moisture survey site R-I-5 located just north of Turtle Mountain, 40 kilometres (25 miles) south of the effective drainage area.

### 2.4.2 Modelling Under Summer Conditions

The model was calibrated using summer data for the years between 1968 and 1978, exclusive of 1977.

Starting lower zone soil moisture levels for each summer were estimated using the data from the fall soil moisture survey and an estimate of spring recharge.

Outflow from HSPF's land segment was passed to the channel routing portion of the model. The basin was divided into seven routing segments as shown on Figure 16. Routing segments one to four contain the headwater segments of the basin with segment four being Elgin Reservoir.

The available cross-section data needed for computing the hydraulic characteristics required for channel routing was quite limited. Cross-sections upstream and downstream of bridge crossings on P.R. 347 indicated channel side slopes of 1:5. Using this slope and a maximum width equal to the bridge openings it was possible to calibrate a rating table to high water marks from the 1976 flood, using a Manning's "n" of 0.030 (Manning, 1891).

#### 2.4.2.1 Calibration

In preliminary runs, simulated runoff exceeded recorded runoff in all years. As infiltration is one of the main processes controlling summer runoff, the parameter which defines the rate of infiltration was the first to be adjusted. This parameter has a range of values of from 0.25 to 1.25 millimetres/hour for clay based soils. During calibration it was found that even when the infiltration parameter was set at the maximum value for the basin's soil type, the reduction in the runoff volume was insufficient.

Another major factor controlling the volume of runoff was the physical size of the watershed. The effective area was initially determined by analysis of the maps and air photos and included areas that could not be positively identified as noncontributing. During calibration the areas of questionable effectiveness, specifically pothole areas in the headwater portion of the watershed, were removed from the effective drainage area. This helped to improve the simulated runoff volumes.

#### 2.4.2.2 Results

Elgin Creek summer model runs were evaluated by visually examining observed and simulated hydrographs and by comparing measured fall soil moistures against simulated lower zone soil moistures. Table 2.6 gives a summary of the final Elgin Creek summer runs. A comparison of the model's October 31 lower zone soil moisture and the observed fall soil moisture showed large differences ranging from 63 percent to 148 percent. The simulated runoff volume for May 1 to October 31 was close to the observed except for the years 1969, 1970 and 1974. However, when the simulated and observed hydrographs were analyzed, the only year for which the simulation was reasonable was 1978. These results are considerably poorer than those for Willow and Gopher Creeks.

Analysis of the hydrographs suggested three major causes for the poor simulation; (1) likely errors in basin area-average precipitation estimates, (2) incorrect assessments of watershed conditions at the start of each run, and (3) nonrepresentative fall soil moistures.

Probable errors in the estimated basin area-average precipitation were detected when the daily precipitation was plotted as an overlay on the simulated and observed hydrographs. This showed that for a large number of events there was no cause and effect relationship between estimated basin area-average precipitation and runoff. For example, a precipitation event of a magnitude expected to produce runoff would occur with no observed runoff and vice versa.

For the other basins studied, errors in starting conditions were corrected by trial and error model runs. In the case of Elgin Creek, recalculation of starting conditions may have improved simulated runoff in May and part of June, but would have had little effect on improving simulations in the summer and fall due to errors in the estimated basin area-average precipitation.

The HSPF model could not be properly calibrated for summer events on Elgin Creek. As discussed previously, calibrations of summer events were undertaken to determine model parameter values and in particular to determine initial spring values for soil and depressional storage parameters. Based on modelling experience, it was estimated that the values determined for actual upper zone storage and upper zone nominal storage could be in error by as much as 13 millimetres and five millimetres respectively.

Table 2.6: Elgin Creek Summer Simulation Results[a]

| Year | May Soil Moisture (mm) | | | | Summer Precip. (mm) | Evapotranspiration[e] (mm) | Runoff (mm) | | Oct. 31 Soil Moisture (mm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | UZS[b] | Fall SMI[c] | + Spring Recharge | = LZS[d] | | | Simulalted | Observed | UZS[b] | LZS[d] | SMI[c] | LZS[b]/SMI[c] (%) |
| 1968 | 12 | 155 | 0 | 155 | 422 | 338 | 23 | | 11 | 188 | 160 | 118 |
| 1969 | 20 | 160 | 25 | 185 | 264 | 312 | 17 | 7 | 1 | 119 | 168 | 71 |
| 1970 | 33 | 168 | 38 | 206 | 262 | 353 | 32 | 19 | 2 | 104 | 135 | 77 |
| 1971 | 13 | 135 | 13 | 148 | 366 | 340 | 9 | 8 | 21 | 135 | 168 | 80 |
| 1972 | 20 | 168 | 0 | 168 | 353 | 386 | 7 | 2 | 7 | 124 | 84 | 148 |
| 1973 | 0 | 84 | 0 | 84 | 411 | 310 | 2 | 5 | 8 | 152 | 196 | 78 |
| 1974 | 28 | 196 | 30 | 226 | 249 | 363 | 33 | 12 | 5 | 109 | 140 | 78 |
| 1975 | 33 | 140 | 38 | 178 | 401 | 404 | 30 | 25 | 23 | 152 | 168 | 90 |
| 1976 | 28 | 168 | 38 | 206 | 180 | 325 | 4 | 7 | 0 | 74 | 117 | 63 |
| 1978 | 13 | 183 | 13 | 196 | 279 | 366 | 4 | 1 | 1 | 97 | 124 | 78 |

**Notes:**

a Results from ELGSUM5.UCI; with infiltration of 1.27 mm/hr, lower zone nominal storage (lzsn) of 203 mm, maximum monthly lower zone evapotranspiration (lzetpm) of 0.54 and monthly upper zone nominal storage (uzsnm) ranging from 20.6 to 33.0 mm.

b UZS is the upper zone storage.

c SMI is the soil moisture index. It is the depth of water measured in the top 430 mm of the soil.

d LSZ is the lower zone storage.

e Combination of plant interception, upper zone evapotranspiration and lower zone evapotranspiration.

### 2.4.3 Modelling Under Spring Conditions

Six median to high flow years were used to calibrate the model for spring runoff.

The model's configuration of the basin was similar to that used for summer simulations with one land segment and seven routing segments.

Estimates of winter snowpack were made by averaging the cumulative basin winter precipitation measured at the three climate stations (Figure 2) for the period from November 1 to February 28. A simple averaging was assumed to be adequate.

#### 2.4.3.1 Calibration

The years selected for model calibration were 1969, 1970, 1974, 1975, 1976 and 1979. Melt was rapid in all of these years, with the lag from the start of spring runoff to the peak discharge ranging between three and five days. The precipitation totals (for both the winter and melt periods) varied from 127 millimetres in 1975 to 185 millimetres in 1974.

Initial runs with calibrated summer process parameters indicated that simulated runoff was much less than observed. HSPF infiltration parameters were modified to account for frozen ground conditions in the same manner as had been done for Willow Creek. This entailed reducing the infiltration rate of the soil as described in Appendix A. The infiltration parameter was adjusted back to the summer value after the spring snowmelt peak.

One of the major calibration difficulties was the determination of the depressional storage parameter, UZSN. Particular emphasis was placed on events occurring in 1969, 1974 and 1976 since these were the result of high fall soil moistures and high winter precipitation. The final calibrated value for the storage parameter was a compromise, which resulted in the best average simulation for all three years. However, runoff for 1969 was overestimated and that for 1974 and 1976 was underestimated. It was discovered later that some of the published Elgin Creek streamflows were estimated for 1969 and could be low by as much as 40 percent. A value of 38 millimetres (1.5 inches) for upper zone nominal storage was used except in 1979 when a value of 64 millimetres (2.5 inches) was used. The 1979 spring event is notable in that a fairly large winter snowpack (177 millimetres) which melted at an unusually fast rate (only three days to peak) resulted in a below average runoff ratio. The low runoff ratio was attributed to the effect of soil cracking. It is postulated that soil cracking caused by the below normal soil moistures in the fall of 1978 caused above normal meltwater absorption in the ground. An adjustment was made to the depressional storage parameter to account for this factor as was done when modelling Willow Creek.

#### 2.4.3.2 Results

Table 2.7 and Figures 17 to 22 show the results of the spring simulation runs for Elgin Creek. Overall the results were not as good as those on Willow Creek which employed the same meteorological data set except for precipitation, and the same method of handling frozen ground conditions. For the Elgin Creek watershed simulated runoff expressed as a percentage of observed runoff showed more variation than those for the Willow Creek or Gopher Creek watersheds, varying between 70 and 120 percent. The timing of simulated spring melt was in error by up to four days. The simulated peak discharge as a percentage of the observed varied between 76 percent in 1976 and 164 percent in 1975.

Use of meteorologic data from Brandon Airport may have hindered model calibration for Elgin Creek because of the problems associated with transposing data over a distance of 50 kilometres. For example ambient and dew point temperatures during the melt were not always representative of the watershed. In April, 1970, data for Deloraine, just south of the effective drainage area, suggested much higher basin temperatures than those computed from Brandon data. This caused simulated flows to lag recorded flows. Synoptic ambient and dew point temperatures were not recorded at Deloraine.

As has been discussed previously, it was not possible to properly model surface depression storage on Elgin Creek and initial values of soil moisture, obtained from the summer simulations, were probably incorrect.

#### 2.4.3.3 Model Verification

For Elgin Creek, as for Willow Creek, all of the major spring runoff events were used during the calibration process. Calibration was so poor that verification runs would not have been justified even if data were available.

Table 2.7: Elgin Creek Spring Summary

| Year | Observed Runoff ( mm ) | Runoff Coef. % | Simulated Runoff ( mm ) | Simulated as % of Observed | Observed Peak ( $m^3$/sec ) | Simulated Peak ( $m^3$/sec ) | Timing |
|---|---|---|---|---|---|---|---|
| 1969 | 35 | 19 | 87 | 249 | 11.3 | 47.0 | 2 days early |
| 1970 | 22 | 23 | 12 | 55 | 9.1 | 4.7 | 4 days late |
|  | 41 | 56 | 44 | 107 | 18.0 | 21.5 | 2 days early |
| 1974 | 108 | 59 | 84 | 78 | 38.5 | 32.0 | 2 days late |
| 1975 | 25 | 19 | 37 | 148 | 15.6 | 17.3 | 1 day late |
|  | 64 | 64 | 55 | 86 | 25.3 | 41.5 | 2 days early |
| 1976 | 120 | 72 | 85 | 71 | 54.4 | 41.5 | 4 days early |
| 1979 | 20 | 12 | 24 | 121 | 16.7 | 17.7 | 2 days early |

**Notes:**

- Results from ELGSPR5.UCI
- Runoff depths are based on the effective drainage area of the watershed.
- Runoff coefficient is the ratio of observed runoff depth to basin area-average precipitation.
- 1970 and 1975 had two distinct peak discharges.

## 2.5 Discussion

This section consists of a general discussion of the major problems and sources of error which are believed to have significantly affected the streamflow simulations for the three watersheds studied. The feasibility of applying and using the HSPF model operationally is also discussed. The simulation results are compared to those obtained using the MANAPI index model currently used by the Manitoba Water Resources Branch.

### 2.5.1 Sources of Error in Hydrologic Modelling

The accuracy of streamflow simulations with the HSPF model is believed to be limited by two major problems. These are inaccuracies in the development of the area-averaged input data and model deficiencies. While inaccuracies in input data also impact on index models, their effect on a complex model such as HSPF appears to be more severe.

#### 2.5.1.1 Hydrometeorological Data

A major source of error in applying the HSPF model to Manitoba watersheds was the nonrepresentativeness of estimated area-average meteorological inputs to the model due in large part to the sparcity of the meteorological station network. Many of the data inputs required for the model had to be estimated from locations outside the study basins.

The objective of the summer modelling was to obtain appropriate parameters for the lands phase of the model and to obtain reasonable estimates of soil moisture at freeze-up preceding the spring calibration years. The accuracy of simulated flows for the summer period varied greatly from basin to basin, year to year and event to event. This variability is thought to have resulted largely from the nonrepresentativeness of basin area-average precipitation estimates. The climatological network is too sparse to give a good areal representation of rainfall amounts and intensities for summer convective storms (Raddatz and Kern, 1984).

The inaccuracy in the estimated basin area-average rainfall reduced the accuracy of soil moisture and streamflow simulations. There were times when significant runoff took place, but little or no precipitation was estimated, indicating substantial errors in basin area-average precipitation. It was postulated that end of summer (October 31) simulations of soil moisture might be reasonably accurate even though simulations of individual runoff events were not. An attempt was made to verify this postulation using field soil moisture measurements taken by the Manitoba Water Resources Branch near freeze-up each fall. While reasonable agreement existed between the simulated and measured soil moistures for Willow Creek, this was not evident for the other two basins. Measured soil moistures, therefore, could not be considered as a sound corroboration of the validity of simulated soil moistures. Furthermore, a single point measurement of soil moisture, may not be representative of an entire watershed and agreement or nonagreement would not prove nor disprove the accuracy of the simulated fall soil moistures.

Spring simulations were thought to be affected less by the nonrepresentativeness of the estimated basin area-average precipitation inputs than summer simulations. The use of seasonal snowfall totals reduces the percent error relative to that for a single snowfall event (Gray and Male, 1981). Nevertheless considerable errors in seasonal snow water content likely remain due to the use of ruler measurements and standard densities at climatological stations and due to the inaccuracies of snow gauge catches under windy conditions. Snowpack estimates were also based on the assumption that overwinter snowpack losses were insignificant. Gray (1979) found that during cold winter months sublimation losses during the day are approximately balanced by condensation gains at night. However, significant net losses may occur during the fall and spring and during unusually mild winter months.

The need to transfer meteorological data for snowmelt simulation from main line stations geographically removed from the basins was a serious source of error in some instances such as in 1970 for Elgin Creek. MacLaren Plansearch (1984) noted that when meteorological data has to be transferred from some distance, or data must be synthesized, the difference in accuracy between the degree-day melt equation and the energy budget approach used in the HSPF model would not be significant.

Inaccuracies in the published streamflow data may have caused modelling errors, since the model was calibrated to these data. Most of the daily streamflow data used for calibration was based on one or occasionally two manual stage observations per day. As such it is difficult to make good estimates of daily flows, especially with rapidly changing stages or ice conditions. Time constraints precluded the use of more than about five years of data for calibration. If a few of these years turn out to be unrepresentative (outliers) due to unusual basin conditions or significant streamflow data errors, the calibrated model parameters may be seriously in error.

In addition, there was virtually no field data describing factors such as soil moisture, plant growth and ponded water to describe the state of the watershed over time. Such data could have provided a means of confirming the calibration of the individual components of the model. Spring modelling might have been improved if data on ground frost were available and if there were observations on snow cover to verify simulated snowmelt.

In addition to inaccuracies in data inputs, there were hydrologic model deficiencies which contributed to errors in streamflow simulation.

HSPF attempts to model the majority of the interactions which compose the hydrologic cycle and as such is considered one of the more sophisticated physically-based models. However, in some instances it still employs simplified relationships to explain complex phenomena. It also lacks the facility to model certain processes explicitly. For example the HSPF model does not contain the capability to model the effect of frozen ground on spring infiltration rates even though frozen soils are known to greatly affect spring infiltration rates.

Difficulties were encountered in modelling the infiltration of meltwater into frozen soil. Modifications which roughly approximated the actual processes were made to the model during the course of this study in an attempt to account for soil frost. The reduction in infiltration rates was approximated by manually changing the infiltration parameter, but this did not account for temporal changes to infiltration characteristics during the melt period.

On the Prairies the basin area contributing to runoff changes with the magnitude of the runoff event. The model does not contain a utility to simulate changes in the effective drainage area from one event to another. This shortcoming may have affected both the summer and spring calibrations.

The original albedo decay function contained in the model was based on observations of changes to albedo in mountainous areas with deep snowpacks. In a mountainous environment it was found that the change in albedo could be accounted for by a time dependant albedo decay relationship. Research has shown that such a relationship is probably inappropriate for shallow snowpacks typically found on the Prairies (Gray, 1979; MacLaren Plansearch, 1984). Although the albedo decay formula was modified in this study (see Appendix A) some questions remain about its applicability.

The validity of the model's computation of evapotranspiration, which is a major component in the summer water balance, is questionable. In the model it is assumed that potential evapotranspiration (PE) is directly related to pan evaporation and that actual evapotranspiration is a direct function of PE for a given soil moisture and stage of plant growth. Research (for example Morton, 1983) suggests that evapotranspiration is in actuality a more complex process which is highly affected by the behavior of plant stomata under various conditions. The calibration of model process parameters and the computation of fall soil moistures were probably negatively affected by inaccuracies in simulated evapotranspiration.

### 2.5.2 Application and Operational Use of HSPF

The application of the HSPF model to the small prairie watersheds examined in this study was relatively time-consuming due to the large quantity of hydrometeorologic input data required and the laborious nature of the calibration process.

It was found that even for a small watershed, where one set of area-average inputs was assumed to be sufficient, it still took an experienced technologist at least a month to prepare the meteorological data files required for application of the HSPF model. Two separate data time series files had to be created; namely, hourly precipitation and daily evapotranspiration for the summer period and synoptic data (six-hour time intervals) for spring snowmelt simulation. Data file preparation was complicated by the sparse meteorologic network, which often required basin data to be estimated from far removed sources. It was also necessary to perform quality control checks on both the raw and the manipulated data. The use of a daily time step for both summer and spring would reduce the complexities of the data files somewhat, but at the expense of simulation accuracy. Elimination of the summer simulations would also reduce the workload, but certain model parameters would be difficult to calibrate on spring data alone.

Calibration of the HSPF model consumed a great deal of time. Before model calibration could begin it was necessary to develop an appropriate basin configuration and to examine maps and aerial photos to obtain physically-based model parameters. Many calibration runs with summer and spring data were required. It was necessary to determine the causes of poor simulations before appropriate corrective action could be taken. This often involved a detailed review of the input data to determine whether it was representative and a review of the model configuration, all of which were very time-consuming. It took an experienced modeler about one month to prepare the model for simulation of a single watershed and an additional two months for calibration. The average time expended on a single basin was, therefore, four man months.

With respect to cost, it was very useful to have an in-house computer. Rental of computer time, as was done during Phase One, would have resulted in prohibitive costs.

HSPF was designed as a generalized water quantity and quality simulation model capable of reproducing the effects of watershed change. It was not specifically designed as a real-time forecast model for spring snowmelt. Very few agencies in Canada or the northern United States had attempted to use HSPF in a forecast mode and documentation of the problems inherent in such an application were not available. Although the HSPF model has not undergone any extensive real-time testing in Manitoba familiarity with the model and with operational forecasting makes it possible to comment on the feasibility of using the model as a forecast tool.

Operational use of the HSPF model would require utility programs to convert meteorological data into the formats required by the model. Real-time estimates of solar radiation would be required. A technique was developed for deriving an estimate of solar radiation based on hourly observed cloud cover, but the procedure was time-consuming and the accuracy of the procedure has not been determined.

Little experience was gained in the real-time adjustment of model parameters in order to achieve a good match (tracking) between recorded and simulated flows. Tracking runs are difficult to do in real time even with less complex models. During a flood event in Manitoba there is usually little data upon which to base adjustments and there are severe time constraints which preclude the use of many trial runs. There is the danger that adjustments which might result in good tracking up to the current time may produce a poor forecast of future flows. With the present meteorological networks it would be necessary to transpose data over considerable distances, thus reducing the accuracy of simulations. Good judgement and experience would be required in order to know whether model parameters or data inputs should be adjusted in a given situation, which parameters should be adjusted, and to what extent the parameters should be adjusted.

It is estimated that with computerized real-time data input an experienced hydrologist might, under typical spring flood conditions, have time to perform tracking runs and to prepare daily forecasts for about a dozen sub-basins.

### 2.5.3 Comparison of HSPF With MANAPI

The primary purpose of this study was to investigate deterministic watershed modelling as a means of improving the forecasting of spring floods. In the preceding sections the performance of HSPF has been compared to observed watershed response. Although the ability of a model to simulate past events is an important evaluation criterion, its performance and utility relative to other models is also important in deciding whether to use it operationally. As such, the accuracy of the HSPF streamflow simulations were compared to those of the model presently used in Manitoba, the MANAPI index model.

A comparison of recorded peaks and runoff depths with those simulated using the HSPF and MANAPI models, for all five of the high flow years available for the study, are shown in Tables 2.8 and 2.9. HSPF spring peak flow simulations were no better, on average, than those of the existing MANAPI index model. The HSPF model simulated runoff volumes were also not consistently better than those simulated using MANAPI, with volumes being simulated more accurately in some years but less accurately in others.

At the start of this study it was thought that a model which explicitly accounts for more of the processes of the hydrologic cycle than the MANAPI index method, would be able to produce better flow forecasts. The results of this study demonstrate that this may not be true given the available inputs. With MANAPI, the complex interactions between basin inputs and outputs are inferred from the past performance of the watershed for the entire period of hydrometric record. The MANAPI method uses an empirical relationship to define basin runoff and an appropriate unit hydrograph to route the water to the streamflow gauge. In this way a complex set of processes are implicitly integrated into a few simple relationships. With a physically-based model, it is necessary to conceptualize and explicitly express many of the processes quite accurately.

The lack of good watershed data had a more serious impact on HSPF than on the MANAPI model. For example, given an appropriate unit hydrograph MANAPI will give a good representation of the combined effects of snowmelt, overland flow, interflow and channel storage for a typical snowmelt pattern even though there may be no information on the physical characteristics of the watershed responsible for these effects. On the other hand, since HSPF attempts to model each of the runoff components separately, data for physical watershed characteristics pertaining to each component is essential. Therefore, HSPF requires a much more extensive quality data network than MANAPI.

Table 2.8: Comparison of HSPF and MANAPI Annual Peak Flow

| Year | | Willow Creek ( $m^3$/sec ) | Gopher Creek ( $m^3$/sec ) | Elgin Creek ( $m^3$/sec ) |
|---|---|---|---|---|
| 1969 | Observed | 77.9 | 17.3 | 11.3[a] |
| | HSPF | 68.7 | 19.4 | 47.0 |
| | MANAPI | 72.5 | 19.1 | 56.3 |
| 1970 | Observed | 58.3 | 8.6 | 18.0 |
| | HSPF | 40.2 | 3.6 | 21.5 |
| | MANAPI | 48.7 | 4.0 | 22.0 |
| 1974 | Observed | 80.4 | 13.4 | 38.5 |
| | HSPF | 69.9 | 19.2 | 32.0 |
| | MANAPI | 88.3 | 20.2 | 57.8 |
| 1976 | Observed | 56.9 | 46.2 | 54.4 |
| | HSPF | 84.0 | 36.7 | 41.5 |
| | MANAPI | 87.8 | 38.3 | 51.0 |
| 1979 | Observed | 71.4 | 10.8 | 16.7 |
| | HSPF | 63.2 | 13.8 | 17.7 |
| | MANAPI | 61.2 | 15.6 | 20.6 |

**Notes:**

a Data is suspect. Peak was likely much greater.

Table 2.9: Comparison of HSPF and MANAPI Spring Runoff Depths[a]

| Year | | Willow Creek (mm) | Gopher Creek (mm) | Elgin Creek (mm) |
|---|---|---|---|---|
| 1969 | Observed | 75 | 65 | 35[c] |
| | HSPF | 68 | 51 | 87 |
| | MANAPI | 68 | 64 | 126 |
| 1970 | Observed | 25[b] | 20[b] | 41[b] |
| | HSPF | 32 | 11 | 44 |
| | MANAPI | 44 | 13 | 49 |
| 1974 | Observed | 82 | 61 | 108 |
| | HSPF | 76 | 56 | 84 |
| | MANAPI | 84 | 67 | 129 |
| 1976 | Observed | 75 | 169 | 120 |
| | HSPF | 75 | 104 | 85 |
| | MANAPI | 83 | 127 | 114 |
| 1979 | Observed | 51 | 40 | 20 |
| | HSPF | 57 | 48 | 24 |
| | MANAPI | 56 | 53 | 46 |

Notes:

a Runoff depths are based on the effective drainage area of the watershed.

b In 1970 there were multiple spring peaks, the volume was only calculated for the largest peak.

c Data is suspect. Runoff was likely much greater.

# Chapter 3

# TRIBUTARY ROUTING STUDIES

## 3.1 Introduction

The ability to simulate the movement of water along stream channels is an important component of hydrologic simulation and flood forecasting. At present, the Manitoba Water Resources Branch uses unit hydrographs to convert predicted runoff from tributary basins to basin outflow hydrographs at the confluence with main streams. The movement of water along the main river channels (Red, Assiniboine and Souris Rivers) is modelled using the Muskingum routing method (McCarthy, 1938).

Although the unit hydrograph procedure is generally adequate for smaller basins, it is less suited to larger tributaries where precipitation or snowmelt may vary from one part of the watershed to another. The nondistributed input which is inherent in the unit hydrograph approach is inappropriate for these watersheds. The basic Muskingum routing method is usually not well suited to routing flows on moderate sized tributaries due to insufficient streamflow gauges and difficulties in determining the routing parameters. This study was initiated during Phase 1 to examine several streamflow routing procedures in order to determine which method might be the most practical and appropriate for tributary routing.

The study also compared the routing methods contained in selected hydrologic simulation models. It was felt that a valid comparison of the runoff simulation portion of the models might be difficult if the routing methods were not equivalent.

The initial phase of this study consisted of an extensive review of the available literature to obtain a better understanding of the various routing procedures. Details of this review are contained in Appendix B of this report.

Five tributary channel routing procedures were examined. These included the One-Dimensional Hydrodynamic (ONE-D) routing model, as well as the simpler Muskingum, Muskingum-Cunge, SSARR, and HSPF routing procedures. After extensive testing of the One-D routing procedure it was concluded that dynamic routing was too complex and difficult to apply to make it viable for routing along small streams with relatively steep slopes.

The four other models concentrate on storage effects as the primary mechanism causing attenuation of the flood wave and replace the dynamic equation of motion with simpler mathematical relationships. These simplified models have potential advantages and disadvantages which are difficult to assess from the literature alone. Therefore, these models were also compared by routing actual flood events down a selected tributary. The tributary had to meet certain requirements. Ideally it should have at least two streamflow gauges and the local inflow between the gauges should be relatively small, when compared to the routed flow. The latter was a requirement because ungauged local inflows are an unknown that prevent a good assessment of routing accuracies. Large in-bank and/or overbank storages between the two gauges were also desirable as this produces significant attenuation. Attenuation of the upstream hydrograph is one of the primary characteristics the routing procedures must simulate.

The reach of the Boyne River between the Stephenfield and Carman streamflow gauges was chosen as the best available location for a comparison of the routing procedures. This reach was not ideal due to significant local inflows during some events. As well some data were missing in the hourly flow records for the higher events. Although streamflow routing using hourly rather than daily values should theoretically produce better flow simulations, hourly data is not commonly available for small streams during the spring forecast period. Due to this fact and because there are missing data in the historic hourly records, it was decided that the routing tests should be conducted using daily discharges.

The basic Muskingum method was subjected to only limited testing since the Muskingum-Cunge method utilizes the same routing equation and an improved method of parameter determination. The passage of several large spring events was simulated with each of the Muskingum-Cunge, SSARR and HSPF methods. Several aspects of the models were examined to determine the model most suitable for routing on tributary streams. The more important of these were: data requirements for model setup, ease of model calibration, computational complexity and the accuracy of simulated flows.

## 3.2 Routing Models

### 3.2.1 One-Dimensional Hydrodynamic Model

A copy of the One-Dimensional Hydrodynamic model was obtained from the Water Planning and Management Branch, Inland Waters Directorate, Environment Canada. The model utilizes the full Saint Venant equations (Saint Venant, 1871) describing one-dimensional flow and had been shown to be very capable of routing floods in large rivers with backwater conditions such as tidal effects and other complicating factors. It was anticipated that this model would provide an accurate means of routing. At the time it was obtained it was also considered for routing output from the hydrologic models.

Trial runs on the Boyne River demonstrated that considerations other than accuracy are equally important to the selection of a routing model. Models which employ an implicit, finite difference computational method have fairly rigid dimensional requirements which are necessary to maintain computational stability and prevent divergent iterations. The requirements are:

1. The spacing between computation points should be approximately seven times the channel width.
2. Rapid transitions between cross-sections in terms of conveyance area, channel slope, etc., require more computation points.
3. The computational time step should be adjusted so that a wave cannot pass between mesh points in less than the modelling period.

Attempts at calibration, even with constant discharges, were difficult. A smooth water surface profile could not be simulated with the cross-sections as originally conceptualized. It became evident that due to the relatively narrow channel width and variable channel cross-section shape, the computational mesh spacing had to be kept short. This, along with the relatively high flow velocities necessitated a computational time step of the order of 60 minutes. Computational costs increased every time the mesh spacing or time step was decreased. Eventually, a decision was reached that the time and computer costs of applying this method made it impractical and, therefore, further work with this model was halted.

### 3.2.2 Muskingum Flood Routing

The Muskingum method of flood routing proposed by McCarthy (1938) has been widely used for many years. It is a hydrologic method and as such concentrates on storage as the primary means of hydrograph attenuation. It is based on a continuity equation which expresses the principle of conservation of mass, in the form:

$$\frac{ds}{dt} = I(t) - O(t) \tag{3.1}$$

where: $S$ is storage, $t$ is time, $I$ is inflow, and $O$ is outflow.

The dynamic equation has been replaced by a simple linear relationship in the form:

$$S_t = K\left[X I_t + (1-X) O_t\right] \tag{3.2}$$

where: $K$ and $X$ are the Muskingum routing parameters.

The parameter $K$ is called the storage time constant and is the ratio of storage within the reach to discharge from it. $K$ is also approximately equal to the ratio between the time of travel of the hydrograph through the reach and the modelling time interval. Increasing the value of $K$ will result in a longer time of storage. The relative importance of inflow and outflow in determining the storage within the reach is defined by $X$ (Meehan and Wiggins, 1979). The volume of water held in storage below a line paralleling the stream bottom is known as prism storage and can be approximated as $KO$. The volume of water contained in storage above this line is known as wedge storage and is approximated by the expression $KX(I - O)$ (Henderson, 1966).

The $X$ parameter ranges from 0.0 to 0.5. A values of 0.5 results in pure translation of the wave (i.e. kinematic wave behavior) corresponding to fully distributed storage, as in a reach whose physical properties are uniform throughout its length. Values of $X$ less than 0.5 introduce a numerical distortion, similar to a diffusive term, which produces an attenuation of the flood wave. A value of 0.0 corresponds to a fully concentrated or reservoir type action where inflow causes an instantaneous response, the principle effect being the attenuation of the inflow peak (Cunge, 1969).

The Muskingum routing equation is commonly expressed as:

$$O_2 = C_1 I_1 + C_2 I_2 + C_3 O_1 \tag{3.3}$$

with the routing coefficients defined as:

$$C_1 = \frac{KX + 0.5(\Delta t)}{K - KX + 0.5\Delta t} \tag{3.4}$$

$$C_2 = \frac{-KX + 0.5(\Delta t)}{K - KX + 0.5\Delta t} \tag{3.5}$$

$$C_3 = \frac{K(1 - X) - 0.5(\Delta t)}{K - KX + 0.5\Delta t} \tag{3.6}$$

Calibration of the Muskingum routing model usually involves the making of initial parameter estimates based on experience and then a trial and error process to determine which routing coefficients produce the best simulation. Several methods of computing the routing parameters can be found in the literature.

### 3.2.3 Muskingum-Cunge Flood Routing

The Muskingum-Cunge flood routing model uses the same routing equation as the original Muskingum model. It differs however, in the method used to compute the routing parameters.

The basic Muskingum model induces attenuation as a result of its finite difference method. This has been termed a mathematical or even an arbitrary diffusion since it is not based on a directly measurable physical parameter. Cunge identified the magnitude of this diffusion with that of the corresponding terms in the convective-diffusion equation. In this way, Cunge was able to develop a methodology for computing the

Muskingum routing parameters based on logically relating the physical characteristics of the channel and the flow in terms of the parameters for the diffusion method. This theoretically allows the attenuation of the peak discharge to be estimated with a high degree of precision while maintaining the computational simplicity of the original Muskingum model.

The theory behind the improvements to the Muskingum model can be found in Cunge (1969) or in the Flood Studies Report (1975) which contains a brief description and Fortran program for applying the method. The computation of the routing parameters using Cunge's method uses $K$ initially defined as equalling the reach length divided by average speed of the flood peak ($\omega$). The derived expression is a finite difference representation of the kinematic wave equation. Further, Cunge observed that by expressing ($I^n$ ) in terms of their Taylor expansions, it could be shown that $X$ could be defined in terms comparable to the diffusion method.

$$\text{when } \mu = (0.5 - X)\,\omega\Delta x \tag{3.7}$$

$$\text{as } \mu = d\frac{\overline{Q_p}}{L} \tag{3.8}$$

$$\text{it follows that X} = 0.5 - \left(\frac{d\overline{Q_p}}{L\omega\Delta x}\right) \tag{3.9}$$

where $L$ is the reach length of the whole reach subdivided into subreaches each of length ($\Delta x$), $d$ is the attenuation parameter corresponding to $Q_p$ (the peak inflow) and $\omega$ is the speed of the flood wave as previously defined. The attenuation parameter is computed using the mean channel width or flooded area, channel length and channel slope.

Once $K$ and $X$ are determined, the routing coefficients are computed using the standard relationships for the Muskingum method.

In order to apply this method over a range of flow magnitudes graphical relationships are developed between peak discharge and the parameters used to compute the routing coefficients so that the modeller can quickly select appropriate values.

### 3.2.4 SSARR Routing Method

The channel routing method used in the SSARR model was developed for modelling the Columbia River basin (Rockwood, 1958) and was called a "multiple-phase reservoir storage channel routing" by the author. The routing method was based on the concept that water input to the basin moves through a series of storage reservoirs which delay basin runoff, starting with the storage on the ground surface and continuing through lake, reservoir, and channel storages. Each of these acts to delay and attenuate the input hydrograph. If the time delay inherent in each of these storage reservoirs could be defined, it would be possible to simulate the lag and attenuation caused by the storages. The channel routing method is applicable to routing where storage is a function of outflow, such as in lakes or short channel reaches, where wedge storage is negligible in comparison with prismatic storage. The SSARR method uses an equation of continuity (Equation 3.1) expressed in the form:

$$I_t = O_t + \frac{ds}{dt} \tag{3.10}$$

The working equation, for computing the outflow at the end of the period ($O_2$) is:

$$O_2 = \left(\frac{I_m - O_1}{T_s + t/2}\right) t + O_1 \tag{3.11}$$

where: $I_m$ = mean inflow, $O_1$ = outflow at the beginning of the period,
$t$ = time duration of computation period, $T_s$ = time of storage.

An important feature in the routing method is that the time of storage ($T_s$) may be varied with the magnitude of flow. The time required for flood waves to travel through a length of stream channel can be made to vary with discharge, according to channel conditions. This allows flexibility in empirically fitting relationships to observed data using constants developed from the actual record. The method may be used to closely represent the actual time delay.

### 3.2.5 HSPF Model

The routing method used in the HSPF model is described by the authors (Johanson et al., 1984) as being an approximation of a kinematic wave model which uses "linear reservoir" concepts to achieve the routing. Kinematic wave models are generally based on the concepts proposed in a paper by Lighthill and Whitham (1955) and commonly use a momentum equation in the form:

$$Q = ah^n \tag{3.12}$$

where: $Q$ is discharge,
$h$ is stage,
and $a$ and $n$ are empirically derived coefficients.

HSPF uses a table to define a variable relationship between depth and discharge. This allows somewhat greater flexibility in defining complex channels but the relationship is still essentially the same as the classic kinematic wave model. A variable storage component is defined in the same table by relating depth to storage volume in the subreach.

Kinematic wave models assume that flood wave movement is primarily kinematic in nature and that dynamic wave movement is insignificant in comparison. As such, discharge and mean flow velocity are assumed to be directly related to flow depth and can be computed using steady state flow equations such as Manning's.

The "linear reservoir" concept can be applied to channel routing using the analogy that a stream channel is like a series of connected lakes or reservoirs. In the case of a reservoir the water surface is assumed to be horizontal and the storage volume is calculated on the basis of horizontal elevation bands. In a stream channel the water surface is assumed to parallel the channel bottom and the volume of storage is calculated by multiplying the average cross-sectional area of flow by the channel length. In both cases it is implied that momentum effects are insignificant in determination of flow characteristics.

In order for this assumption to be physically valid for channel routing, where momentum effects are known to be much more important, the model must be configured in a manner to minimize dynamic wave characteristics. The reach lengths must be short enough that wedge storage (the water contained in the dynamic disturbance) is insignificant in comparison to prism storage.

As dynamic wave effects will no longer be significant within each reach, a rating curve can be computed using steady state equations which will contain a single value of discharge and volume of storage for each increment of stage.

The HSPF model uses the equation of continuity expressed in terms of volumes:

$$Vol_2 \;=\; (inflow\ volume + Vol_1) - outflow\ volume \tag{3.13}$$

where: Vol is the volume of water in storage at time 1
Vol is the volume of water in storage at time 2

The principle task of the routing is to estimate the outflow volume and hence $Vol_2$. The outflow volume is estimated as:

$$outflow\ volume \;=\; [KS * Qout_1 + (1 - KS)\,Qout_2]\,t \tag{3.14}$$

where: KS is a weighting factor (between 0 - 0.99)
$Qout_1$ is the outflow rate at the start of the period
$Qout_2$ is the outflow rate at the end of the period

The mean rate of outflow is considered to be the weighted average of the rates at the start and end of the interval. The weighting factor can be used to bias the outflow rate to either the start or end of the period. This is applied in cases where trial routings indicate a need for some adjustment of the routing parameters, but where there is no justification or information for changing the table entries. The weighting factor must be selected with care. The risk that the computation of outflow rates will become unstable increases as $KS$ increases. A value of 0.5 gives the most accurate results, provided that oscillations do not occur, while the default value of 0.0 has zero risk but gives less accurate results.

When the model is used for channel routing alone the rate of outflow at the end of the period can be expressed as:

$$Qout_2 = funct(Vol_2) \tag{3.15}$$

Routing through each reach can be accomplished by the simultaneous solution of Equations 3.13 and 3.15. A subroutine in the model is provided for this.

The application of the HSPF routing subroutine requires the computation of reach surface area and storage volume for each depth increment. This is usually accomplished using surveyed cross-section data. Discharges for a given depth can either be the observed discharge at a gauge location or the computed discharge for an end of reach location based upon the survey information and a steady state equation such as Manning's or Chezy's. The model makes no assumptions about channel shape, but requires that the functional relationship be constant in time. The weighting factor $KS$ is altered only when trial routings indicate that fine tuning is necessary.

### 3.2.6 Comparison of Model Routing Theories

The theoretical differences between the simpler flood routing models often becomes obscured in the mire of technical jargon common in this field. Essentially the four simpler methods can all be described as hydrologic models since they all concentrate on the effect of storage in attenuating the peak discharge. The significant differences between them lies in the manner in which the routing parameters are determined and whether they are variable during the passage of the flood wave.

The basic weakness of the Muskingum and Muskingum-Cunge methods of routing for tributary flows is that they use one set of routing parameters throughout the modelling period. As such, routing parameters which will accurately simulate the initial rise of the hydrograph are likely to cause error in the simulation of the peak discharge. Similarly, if the routing parameters are adjusted to simulate the peak discharge, the routed hydrograph will not track properly before the peak. This problem exists for any modification of the Muskingum method, except those which recalculate the routing parameters for each routing period. The Manitoba Water Resources Branch has attempted to alleviate this shortcoming by manually intervening and reinitializing the routing, with new values for $K$ and $X$, for different stages of flow. While this causes discontinuities in the routed hydrograph, it is more appropriate than the use of one set of $K$ and $X$ values throughout.

Another drawback of the Muskingum method is that the two routing parameters are most commonly estimated by the trial and error routing of recorded streamflows. This method cannot be applied where there is insufficient streamflow data. This has led to the observation by some authors that the attenuation induced by the model is an arbitrary diffusion since it cannot be directly related to stream channel characteristics (Miller and Cunge, 1975). This makes the application of the method beyond the range of observed streamflows extremely risky.

The Muskingum-Cunge procedure has the capability to simulate the attenuation and timing of the flood peak with a precision similar to the convection-diffusion equation. The routing parameters are computed using data on stream channel and flow characteristics and thus are not subject to a trial and error calibration, if there is adequate data. In addition, the routing parameters can also be extrapolated beyond the range of observed flows with more confidence than the basic Muskingum method.

Two main drawbacks exist with the Muskingum-Cunge method. The first is that it requires stream channel cross-section data to be able to define the routing parameters and this data is often not available. Secondly, the method is still commonly applied using one set of routing coefficients for the entire hydrograph. This is due to the simpler mathematics involved when using constant routing parameters, which allows the use of hand calculations. The hydrograph before and after the peak will still be distorted in the same manner as with the original Muskingum method.

The routing method used by the SSARR model has the advantage that the proportionality factor ($T_s$) which describes the reach travel time is variable with discharge. This variability is usually defined by a power function. The fundamental routing equation can be shown to be the Muskingum routing equation with $K = T_s$ and $X = 0$ (Miller and Cunge, 1975). This value of $X$ relates to a reservoir type of routing and indicates that the maximum possible attenuation is obtained from the formula and no further calibration of the storage parameter is possible. The amount of attenuation is reduced by increasing the number of reaches the flow must pass through, while keeping the total time of storage constant. This method is similar to routing methods which modify the flood wave by altering the number of phases the wave must pass through (Tatum, 1940) and to level pool routing methods such as Puls (1938).

A major drawback to the SSARR method is the inflexibility of the storage parameter. This limits not only the calibration range but also the calibration method. Even if the storage times for different size floods can be identified from observations and the correct proportionality equation derived, the amount of attenuation must still be adjusted using a trial and error process to determine the number of routing phases. As the storage parameter is constant the amount of attenuation can only be adjusted by altering the number of routing phases.

As with the SSARR routing, the advantage of the HSPF method is that the routing parameters change continuously throughout the routing in response to changes in flow magnitude. The HSPF method has the further advantage of not requiring a trial and error calibration procedure. A disadvantage of this method is that the stream channel data required to be able to define the stage-storage and stage-outflow relationships is not always available and is expensive to obtain.

## 3.3 Routing Trials

### 3.3.1 Methodology

A routing comparison was performed using the Muskingum-Cunge, SSARR and HSPF methods. The perspective was taken that the models would be applied in a manner similar to that which would be used operationally. This limited the inflow hydrograph to a single discharge value per day. In addition, the Water Survey of Canada streamflow records were taken to be correct and no attempt was made to verify the recorded flows. There was also no attempt to correct for lateral inflows.

The amount of time and data required for calibration varied considerably between the routing methods. Muskingum-Cunge required extensive data on stream channel cross-sections and streambed slope in order to define the attenuation parameter. Recorded streamflows at the upstream and downstream locations were used to define the time of travel for two flood events.

The SSARR routing method was applied using observed flows and a trial and error calibration process. This method of calibration was tedious and did not instill confidence in the derived relationship.

The routing method contained in HSPF was applied using cross-section information to compute the stage/storage relationship. The discharge component of the routing relationship was defined by integrating the available rating curves into the tables. Storage volumes were computed by multiplying the cross-sectional area by the stream channel length. The derived tables containing the computed volumes and interpolated rating tables were used with no attempt at calibration.

### 3.3.2 Results

Six springtime flood hydrographs on the Boyne River between Stephenfield and Carman were routed using daily flows. The inflow hydrographs, observed downstream flows and routed flows are presented in Figures 23 to 28.

A numerical comparison of the trial routings was not performed. This type of analysis requires accurate flow records including the contribution by lateral inflows in order to be valid. In this reach, the total volume of flow which is unaccounted for (i.e. inflow volume - outflow volume = local inflows and/or errors in the recorded streamflows) is of the same order of magnitude as the differences between the routed flows. For example, the published streamflow record for 1974 indicates that the peak outflow from the reach was 18.1 $m^3$/sec (640 $ft^3$/sec ) higher than the recorded inflow. If the streamflow records are correct this would indicate that the local inflow, at the time of the outflow peak was at least 18.1 $m^3$/sec . Since the local inflow was not accounted for in the routings, all of the routing methods under-simulated the outflow peak by at least 21.0 $m^3$/sec . At the same time the difference between the highest and lowest routed peaks was only 6.9 $m^3$/sec (240 $ft^3$/sec ). In such a case, the numerical comparison of flows would not serve any useful purpose.

While a numerical comparison was not done, several trends were identified from a qualitative analysis of the results.

1. The attenuation and lag of the peak discharge simulated by the three methods was very similar in almost all years, while Muskingum-Cunge induced significantly more attenuation in 1969 and 1979.
2. The SSARR method of routing produced outflow hydrographs which track somewhat better in most years than the other methods.
3. The HSPF method lagged the flows consistently less than the other methods, especially at lower discharges such as in 1969.

The hydrographs simulated by the Muskingum-Cunge method display some unrealistic distortion, as a result of using constant values of K and X to route the entire hydrograph. However, for most years the simulated lag and attenuation of the peak discharge were similar to those produced by the other methods.

The advantage of a variable parameter method is illustrated by the results produced by the SSARR routing method. This method generally reproduced the observed hydrograph better than the other methods.

The HSPF method lagged the flows consistently less than the other methods, especially at lower discharges such as those recorded in 1969. This was attributed to the use of rating curves to derive the discharge value for the routing tables. Streamflow gauge locations tend to be at relatively clean and stable channel sections which might not be representative of the flow characteristics generally along the entire reach. As such the stage/discharge/storage relationship overestimated the hydraulic conductivity of the reach and under-simulated lag times. Since the routing tables were derived with virtually no attempt at calibration it would seem likely that this could be easily corrected.

## 3.4 Conclusions

It is believed that all three models could be used to simulate the lag and attenuation of the peak discharge with similar accuracy. The major differences between the models appears to be in the ease of application. The method used in the SSARR model requires only observed flows to estimate the routing parameters and calibrate the model. Unfortunately, calibration of this model requires repetitive trial runs and accurate flow records from streamflow gauges located at the upstream and downstream ends of the reach.

The Muskingum-Cunge method simulated the magnitude and timing of peak flows with an accuracy similar to that produced by the other methods. A variable parameter version of this model might, therefore, be expected to simulate the entire hydrograph with an accuracy similar to the SSARR method. The major drawback to this method is that it requires the largest amount of data for the derivation of the routing parameters. Both stream channel data and observed hydrographs are used to compute the attenuation parameter and determine the flood wave speed. These must be derived for at least two events in order to even crudely define the necessary graphical relationships.

The flows simulated by the HSPF method were generally about as accurate at the peak as the other methods, but did not track as well as SSARR at lower discharges. This can be attributed to the lack of model calibration. Since rating tables were available, it was assumed that only the storage volumes needed to be computed and the model could be run with no calibration. This was successful to an extent, as evidenced by the routing results. Errors were introduced by assuming that the rating tables could be used to represent the discharge characteristics for the entire reach. The use of steady state formulae for a greater number of locations would allow greater flexibility in the calibration process and should produce results as accurate as those of the SSARR model.

Unknowns in the local ungauged flows together with possible errors in the published flows for the two hydrometric stations used in the routing study, undoubtably reduced the reliability of the study results. However, the lack of major differences between the routed flows for the various methods suggests that the method easiest to apply should be used. This may vary from one watershed to another depending on the availability of data on streamflows and channel characteristics.

# Chapter 4

# RECOMMENDATIONS

This study has shown that the HSPF model does not provide significantly better simulations of runoff volumes and peak flows than the MANAPI index model currently used for flow forecasting in Manitoba. The HSPF model could be used to advantage in situations where conditions are discontinuous, such as when snowmelt is interrupted by a cold snap, or when a major rainfall occurs soon after spring runoff is underway. However, these situations do not arise very often in Manitoba. Therefore, the expenditures in manpower and time required for the wide application of the HSPF model in Manitoba cannot be justified at this time. Present hydrometeorological networks are too sparse to enable effective use of the model and the model itself has shortcomings. Operational use of the model would require manpower which may not be available in the Manitoba Flow Forecast Centre when this Agreement expires.

It is, therefore, recommended that:

1. Application of the HSPF hydrologic model for flow forecasting in Manitoba be discontinued.

2. Lumped index models such as MANAPI or improvements thereof continue to be used for river forecasting in southern Manitoba until more accurate and practical simulation models become available.

3. An operations manual should be prepared for the Centre, outlining the various flow forecasting procedures which have been developed and are being used operationally.

4. The application of streamflow forecasting methodologies be streamlined through increased computerization where feasible.

5. Hydrometeorological data networks continue to be improved to enable the updating and efficient implementation of flow forecasting procedures. Emphasis should be placed on continuing to improve the quality of streamflow data and on increasing the density of precipitation and hydrometric networks.

6. The availability of hydrometeorological data in real-time should be improved. Emphasis should be placed on installing more real-time data collection and telemetry systems to continuously monitor water levels on streams and lakes and on increasing the number of climatological stations reporting precipitation on a daily basis. Tipping bucket or Belfort precipitation gauges should be colocated with hydrometric real-time stations wherever feasible to provide additional data on precipitation amounts and intensities.

7. Methods of obtaining better information on the spacial distribution of precipitation, soil moisture, snowcover and flooded areas using remote sensing devices be investigated and implemented, where they can be shown to be a net benefit to streamflow forecasting.

8. Rivers and streams where the present streamflow routing procedures are inadequate should be identified. In these cases, more appropriate routing models utilizing physical channel characteristics should be applied where potential benefits outweigh incremental costs.

# Bibliography

[1] Canada-Manitoba Flood Damage Reduction Program (1985), *Agreement Respecting Flood Forecasting; Phase One, Planning and Design* (unpublished).

[2] Cunge, J. A., (1969), "On the Subject of a Flood Propagation Method", *Journal of Hydraulics Research*, IAHR, 7, pp. 205-230.

[3] CH2M Hill, (1980), *Analysis of Existing Hydrologic Models, Red River of the North Drainage Basin North Dakota and Minnesota* Report submitted to the St. Paul District of the U.S. Army Corps of Engineers, St. Paul, Minnesota, pp. 3-32.

[4] Donigian, A. S., Jr. and Davis, H. H., Jr. (1978), *User's Manual for Agricultural Runoff Management (ARM) Model.* U.S. Environmental Protection Agency. Athens, Georgia. EPA-600/3-78-080.

[5] Granger, R. J., Gray, D. M. and Dyck, G. E., (1984). "Snowmelt Infiltration to Frozen Prairie Soils", *Canadian Journal of Earth Sciences*, 21(6), pp. 669-677.

[6] Gray, D. M., (1979), *Snow Hydrology Studies of Basin Accumulation and Ablation Processes in the Context of Hydrologic Modelling in the Prairie Environment*, Division of Hydrology, University of Saskatchewan, Saskatoon, Saskatchewan (internal report).

[7] Gray, D. M. and Male, D. H. (1981), *Handbook of Snow, Principle, Processes, Management and Use*, Pergamon Press, Toronto, Ontario.

[8] Gray, D. M., Landine, P. G. and McKay, G. A., (1985), *Forecasting Streamflow Runoff from Snowmelt in a Prairie Environment*, Proceedings from the Canadian Society of Civil Engineering, Saskatoon, Saskatchewan, pp. 213-231.

[9] Henderson, F. M., (1966), *Open Channel Flow*, Mcmillan, New York, New York.

[10] Hershel, Clemens, (1897), "On the Origin of the Chezy Equation", *Journal, Association of Engineering Societies*, vol. 18, pp. 363-368. Discussion, pp. 368-369, January-June, 1897.

[11] Hydrocomp Incorporated, (1977), *Agricultural Runoff Management (ARM) Model- Version II, Refinement and Testing*, U.S. Environmental Protection Agency, Athens, Georgia. EPA-600/3-77-098.

[12] Johanson, R. C., et al, (1984), *Hydrological Simulation Program - Fortran (HSPF): User's Manual for Release 8.0*, EPA-600/3-84-066, Environmental Research Laboratory, Office of Research and Development, USEPA, Athens, Georgia.

[13] Lighthill, M. J., and Whitham, G. B., (1955), "On Kinematic Waves. I-Flood Movement in Long Rivers", *Proceedings, Royal Society of London*, Series A, 229, pp. 281-316.

[14] MacLaren Plansearch (1984), *Snow Hydrology Study Phases I and II: Study Methodology and Single Event Simulation.* Report submitted to Conservation authorities and Water Management Branch, Ministry of Natural Resources, Province of Ontario, Toronto, Ontario, pp. 8-4.

[15] Manning, R. (1891), "On The Flow of Water in Open Channels and Pipes", *Trans. Inst. Civil Engrs. of Ireland*, Dublin, 1891, pp. 161-207.

[16] Manitoba Department of Agriculture, (1983), *1983 Manitoba Agriculture Yearbook*, Manitoba Department of Agriculture, Winnipeg, Manitoba, pp. 112.

[17] McCarthy, G. T., (1938), "The Unit Hydrograph and Flood Routing" paper presented at the Conf. of the North Atlantic Div. of the U.S. Corps of Engineers, New London, Connecticut.

[18] Meehan, P. J. and Wiggins, N. C., (1979), "The Last of the Muskingums", *Proceedings of the Fourth National Hydrotechnical Conference - River Basin Management*, CSCE, Vol. 1, pp. 332-345.

[19] Miller, W. A. and Cunge, J. A., (1975), "Simplified Equations of Unsteady Flow", in Mahmood, K. and Yevjevick, V., (eds.) *Unsteady Flow in Open Channels*, Vol. I, Fort Collins, Colorado: Water Resources Publications.

[20] Morton, F. I., (1983), "Operational Estimates of Areal Evapotranspiration and Their Significance to the Science and Practice of Hydrology". *Journal of Hydrology*, Volume 66, pp. 1-76.

[21] Natural Environment Research Council, (1975), *The Flood Studies Report, Volume III, Flood Routing Studies*, Whitefriars Press, London.

[22] PFRA, (1983), *The Determination of Gross and Effective Drainage Areas in the Prairie Provinces*, Hydrology Report No. 104, Hydrology Division, Regina, Saskatchewan.

[23] Puls, L. G., (1928), "Construction of Flood Routing Curves", *House Document 185, U.S. 70th Congress, 1st Session*, Washington, DC, pp. 46-52.

[24] Raddatz, R. and Kern, J. (1984), "An Assessment of the Near Real-Time Rainfall Network of Canada's Eastern Prairies", *Atmosphere Ocean*, Volume 22, April, 1984, pp. 474-483.

[25] Rockwood, D. M., (1958), "Columbia Basin Stream Flow Routing Computer", *Transactions*, ASCE, 126(4), pp. 32-56.

[26] Saint-Venant A.C. Barre de, (1871), "Theorie du mouvemnt non permanent des eaux, avec application aux crues des rivieres et a l'introduction des marees dans leur lits", (Theory of the nonpermanent movement of waters with application to the floods of rivers and to the introduction of the tides within their beds), *Comptes rendus des seances de l'Academie des Sciences*, Vol. 73, pp. 147-154 and 237-240.

[27] Tatum, F. E., (1940), *A Simplified Method of Routing Flood Flows Through Natural Valley Sturage*, (unpublished memorandum), U.S. Engineer's Office, Rock Island, Illinois.

[28] Thiessen, A. H. (1911), "Precipitation for large areas", *Monthly Weather Review*, Vol. 39, July, 1911, pp. 1082-1084.

[29] U.S.A.C.E. (1956), *Snow Hydrology, Summary Report of the Snow Investigations*, North Pacific Division, Corps of Engineers, U.S. Army, Portland, Oregon.

[30] Warkentin, A. A., (1986), *River Flow Forecasting Procedures Employed by the Manitoba Flood Forecast Centre, (draft report), Manitoba Water Resources Branch, Winnipeg, Manitoba.*

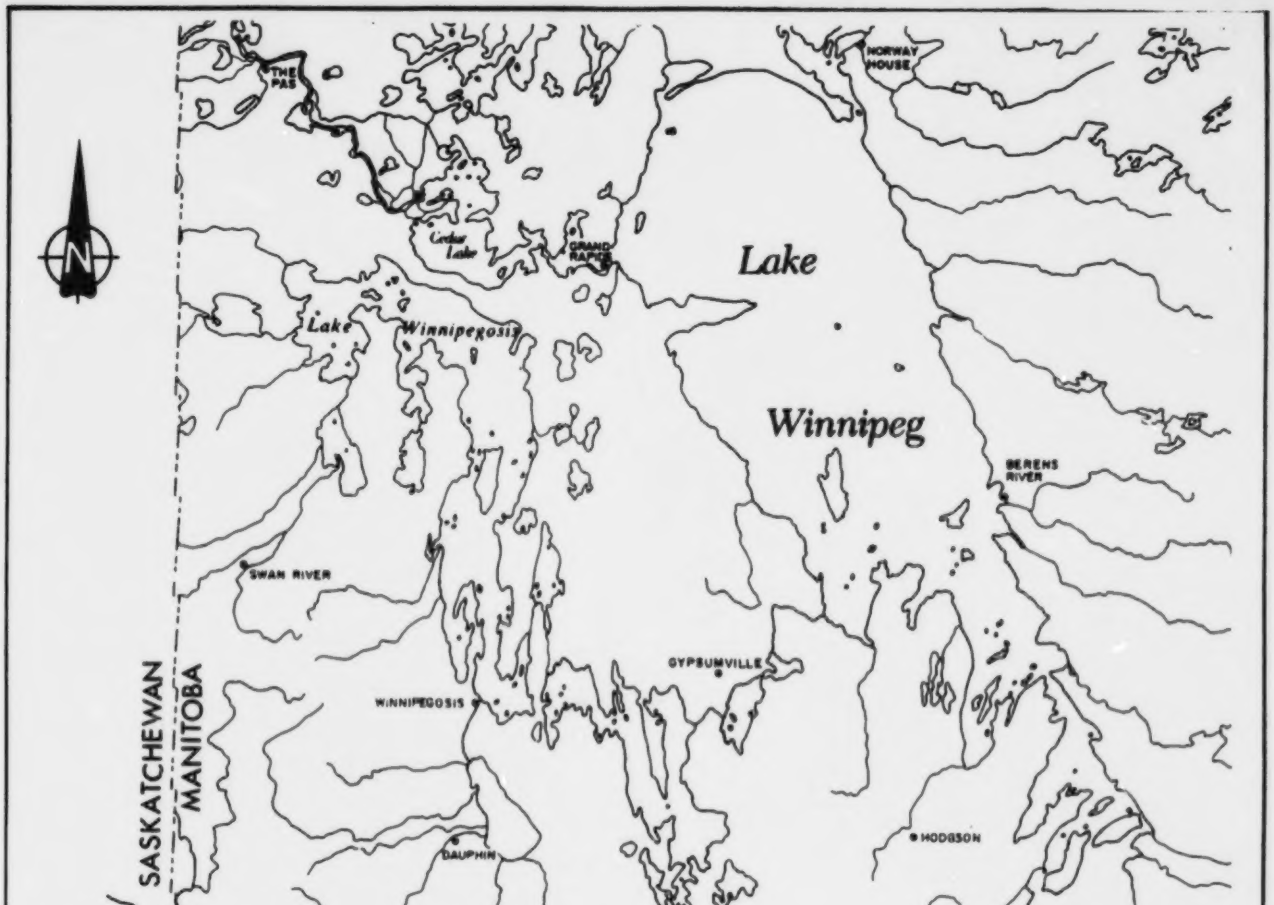

THE PAS
NORWAY HOUSE
Cedar Lake
GRAND RAPIDS
Lake Winnipeg
Lake Winnipegosis
BERENS RIVER
SWAN RIVER
GYPSUMVILLE
WINNIPEGOSIS
SASKATCHEWAN
MANITOBA
HODGSON
DAUPHIN
RUSSELL
McCREARY
GIMLI
Lake Manitoba
SHOAL LAKE
MOOSOMIN
FLEMING
GOPHER CREEK
NEEPAWA
SELKIRK
MARYFIELD
PORTAGE LA PRAIRIE
WILLOW CREEK
WINNIPEG
VIRDEN
BRANDON
Assiniboine
Red River
GLENBORO
STEINBACH
Souris
ELGIN CREEK
MORRIS
DELORAINE
Pembina River
MORDEN
KILLARNEY
EMERSON
CANADA
U.S.A.
0 10 20 30 40 50 MILES
0 20 40 60 80 KILOMETRES

FIGURE 1

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

# LOCATION PLAN OF WATERSHEDS MODELLED

DRAWN J. MAMOTT
CHECKED
PREPARED E. BLAIS

Manitoba
Natural Resources
Water Resources

SUBMITTED
APPROVED
SCALE AS SHOWN
DATE 86 05 06
SHEET OF
FILE NO.

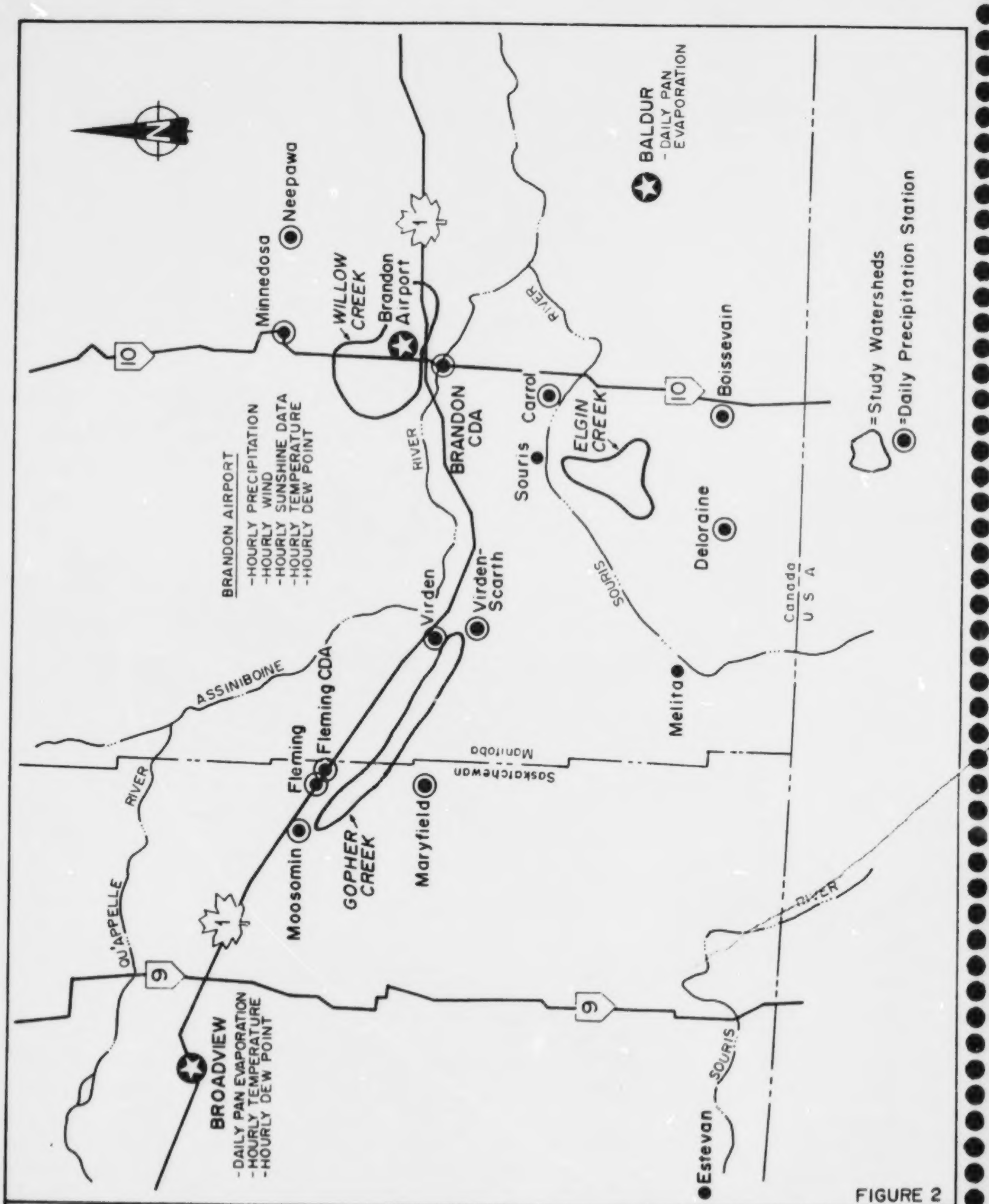

N
BRANDON AIRPORT
-HOURLY PRECIPITATION
-HOURLY WIND
-HOURLY SUNSHINE DATA
-HOURLY TEMPERATURE
-HOURLY DEW POINT
BROADVIEW
-DAILY PAN EVAPORATION
-HOURLY TEMPERATURE
-HOURLY DEW POINT
BALDUR
-DAILY PAN EVAPORATION
Neepawa
Minnedosa
WILLOW CREEK
Brandon Airport
BRANDON CDA
10
1
9
Carrol
Souris
Boissevain
ELGIN CREEK
Deloraine
Melita
Estevan
Virden
Virden-Scarth
Fleming
Fleming CDA
Moosomin
GOPHER CREEK
Maryfield
ASSINIBOINE
RIVER
QU'APPELLE
RIVER
SOURIS
RIVER
Saskatchewan
Manitoba
Canada
U S A
=Study Watersheds
=Daily Precipitation Station

FIGURE 2

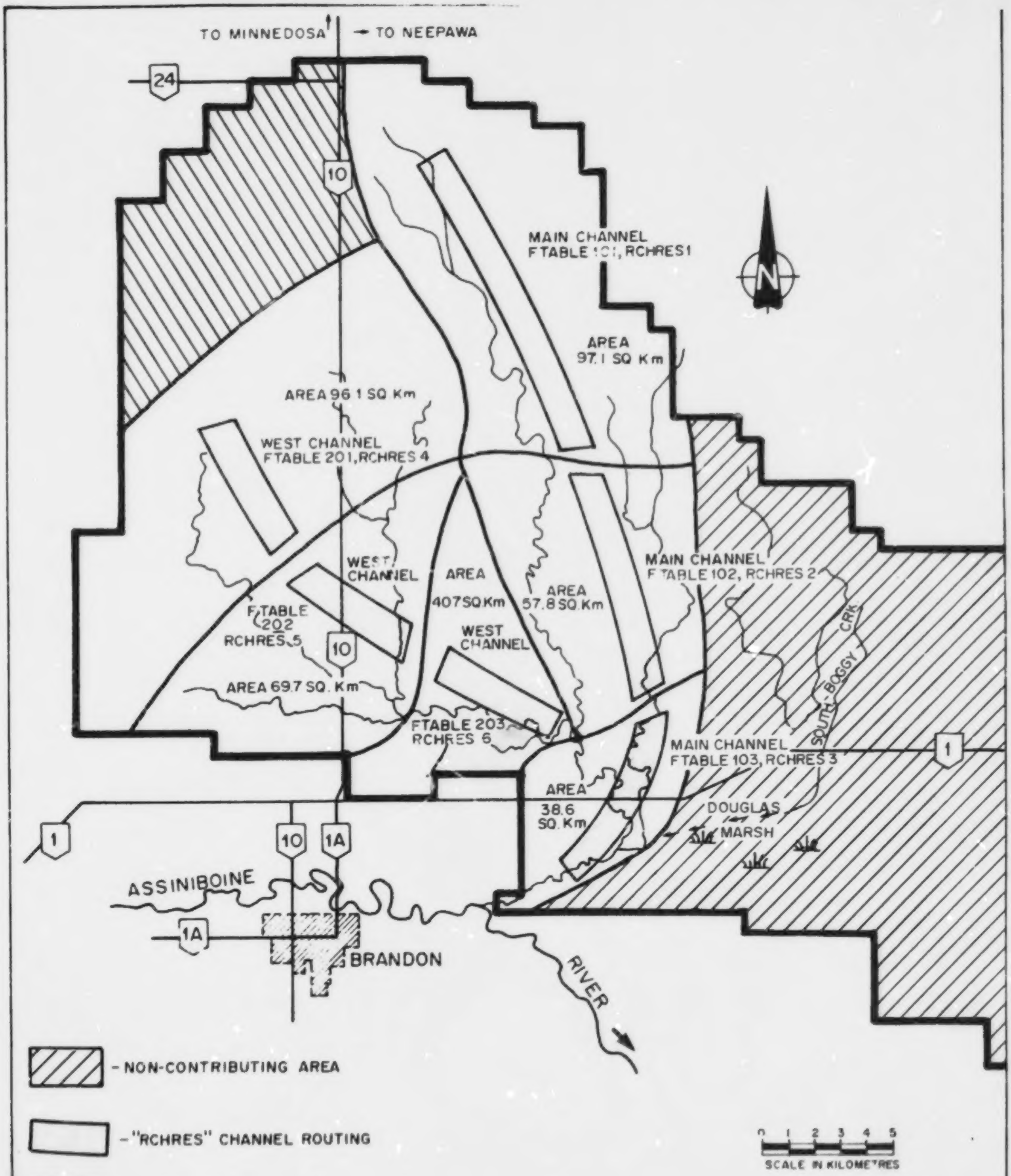
DRAWN
J. MAMOTT
CHECKED
PREPARED
G. MOHR

**Manitoba Natural Resources**
Water Resources

SUBMITTED
APPROVED

# METEOROLOGICAL NETWORK

SCALE AS SHOWN
DATE 87 01 02
SHEET OF
FILE NO.


TO MINNEDOSA
TO NEEPAWA
24
10
MAIN CHANNEL
FTABLE 101, RCHRES1
N
AREA
97.1 SQ Km
AREA 96.1 SQ Km
WEST CHANNEL
FTABLE 201, RCHRES 4
WEST CHANNEL
AREA
40.7 SQ Km
AREA
57.8 SQ Km
MAIN CHANNEL
FTABLE 102, RCHRES 2
FTABLE
202
RCHRES 5
10
WEST
CHANNEL
SOUTH BOGGY CRK
AREA 69.7 SQ. Km
FTABLE 203
RCHRES 6
MAIN CHANNEL
FTABLE 103, RCHRES 3
1
AREA
38.6
SQ. Km
DOUGLAS
MARSH
1
10
1A
ASSINIBOINE
1A
BRANDON
RIVER
0 1 2 3 4 5
SCALE IN KILOMETRES

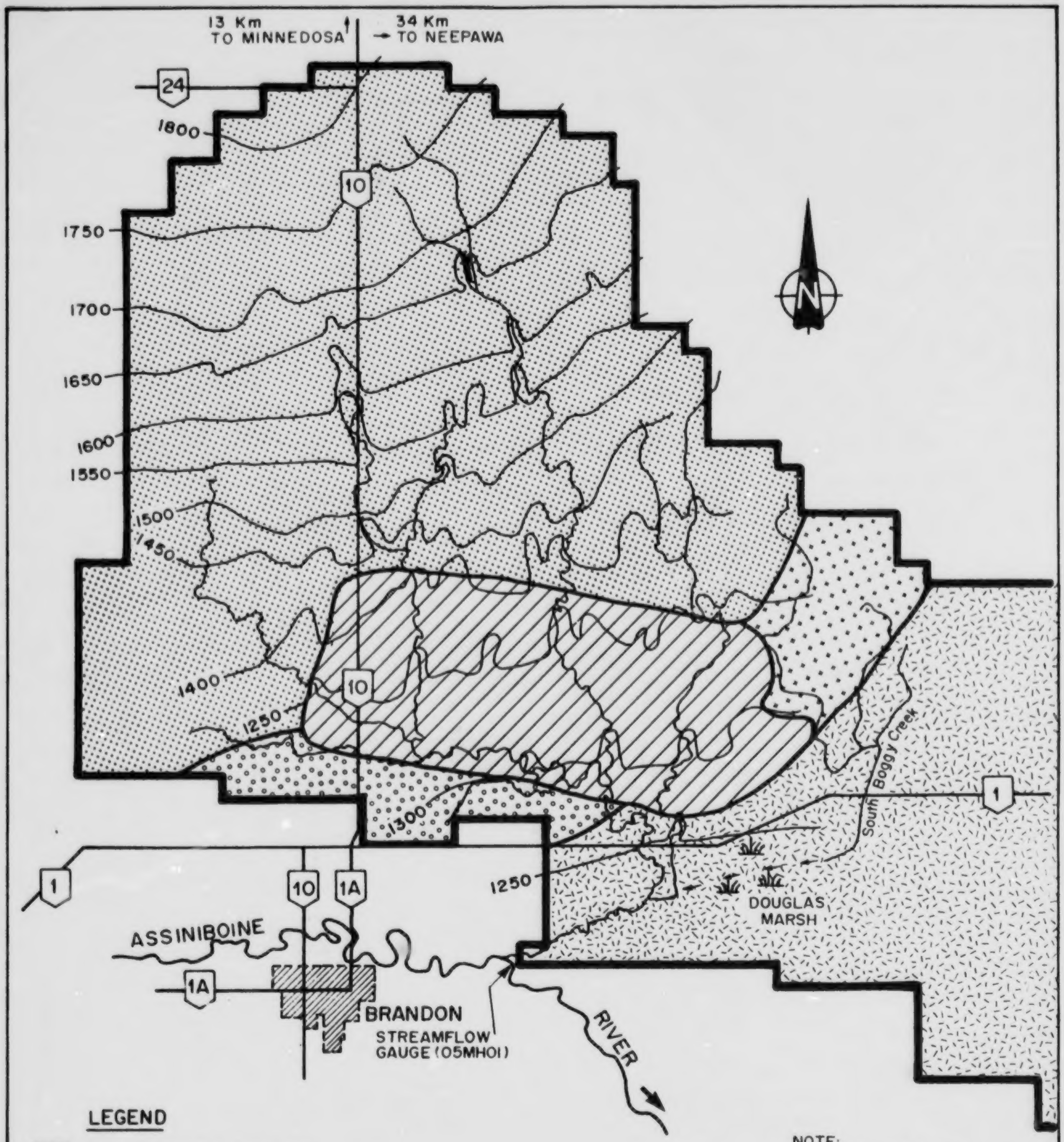
▨ - NON-CONTRIBUTING AREA

▭ - "RCHRES" CHANNEL ROUTING

▭ - CONTRIBUTING AREA

FIGURE 3

| | | | |
|---|---|---|---|
| DRAWN<br>J MAMOTT | **Manitoba Natural Resources**<br>Water Resources | | CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM<br>FLOOD FORECASTING AGREEMENT-PHASE TWO<br>WILLOW CREEK WATERSHED<br>HSPF CONFIGURATION |
| CHECKED | | | |
| PREPARED<br>G MOHR | SUBMITTED | APPROVED | SCALE AS SHOWN · DATE 86 01 21 · SHEET OF · FILE NO |


13 Km
TO MINNEDOSA
34 Km
TO NEEPAWA
24
1800
10
1750
1700
1650
1600
1550
1500
1450
1400
1250
1300
N
South Boggy Creek
1
10
1A
1250
DOUGLAS MARSH
ASSINIBOINE
1A
BRANDON
STREAMFLOW GAUGE (05MH01)
RIVER

**LEGEND**

- ▨ - TILL DEPOSIT, 1-10m THICK
- ▨ - DEEP BASIN DEPOSIT, CLAY + SAND, 1-30 m THICK
- ▨ - BEACH DEPOSIT, SAND + GRAVEL, 1-4m THICK
- ▨ - TILL DEPOSIT, HIGHLY CALCAREOUS, MORAINE
- ▨ - DELTAIC DEPOSITS

NOTE:
ELEVATION IN FEET

0 1 2 3 4 5
SCALE IN KILOMETRES

FIGURE 4

| | | | |
|---|---|---|---|
| DRAWN<br>J. MAMOTT | Manitoba<br>Natural Resources<br>Water Resources | | CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM<br>FLOOD FORECASTING AGREEMENT-PHASE TWO<br>WILLOW CREEK WATERSHED<br>SURFICIAL GEOLOGY AND RELIEF MAP |
| CHECKED | | | |
| PREPARED<br>G. MOHR | SUBMITTED. | APPROVED | SCALE AS SHOWN · DATE 86 01 21 · SHEET OF · FILE NO. |

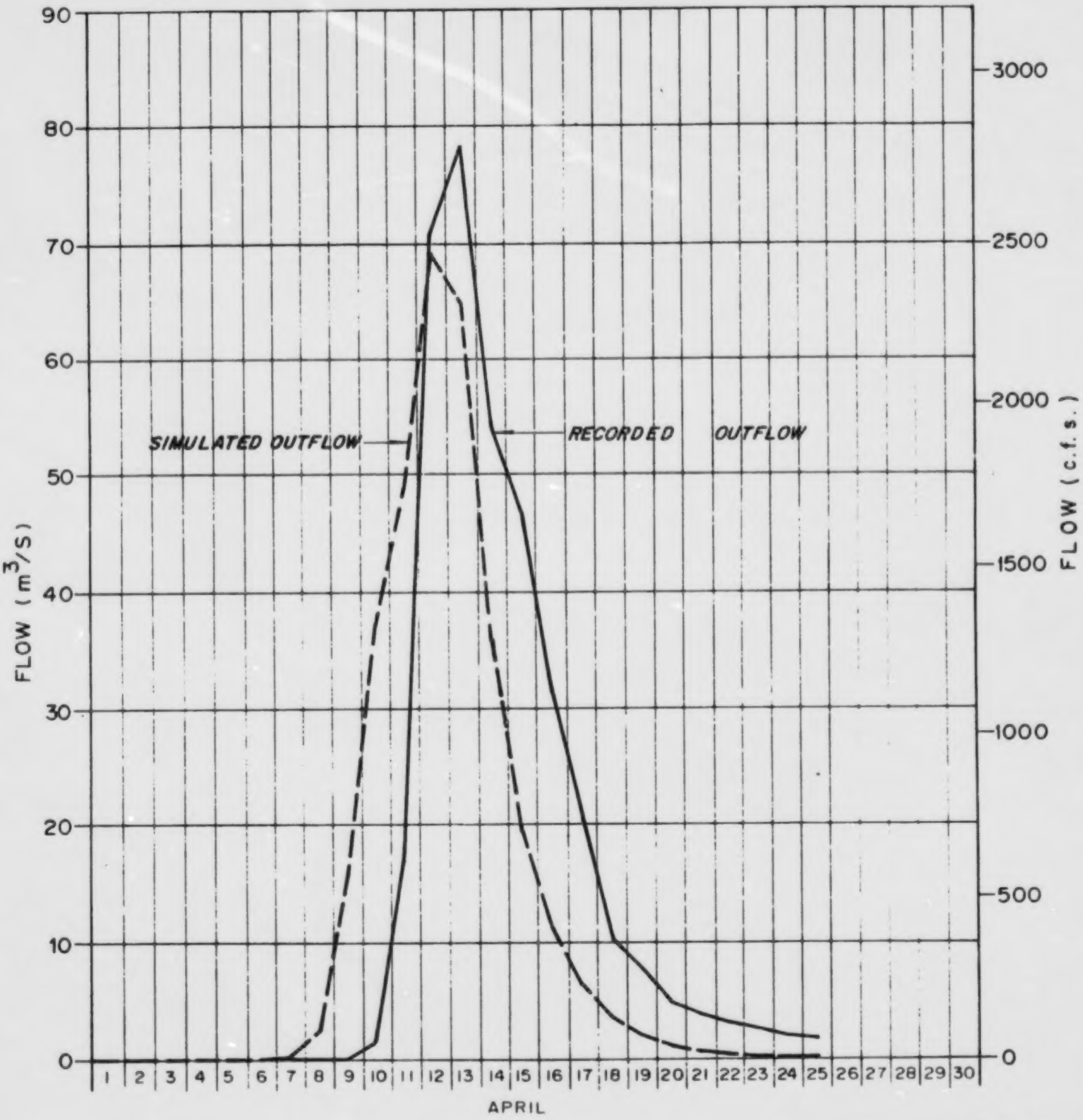


FIGURE 5

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

**Manitoba Natural Resources**
Water Resources

## WILLOW CREEK FLOWS 1969

| | | | | | | |
|---|---|---|---|---|---|---|
| DRAWN M. M. | CHECKED | PREPARED G. MOHR | SUBMITTED | APPROVED | | |
| SCALE AS SHOWN | DATE 87 01 14 | SHEET OF | FILE NO | | | |

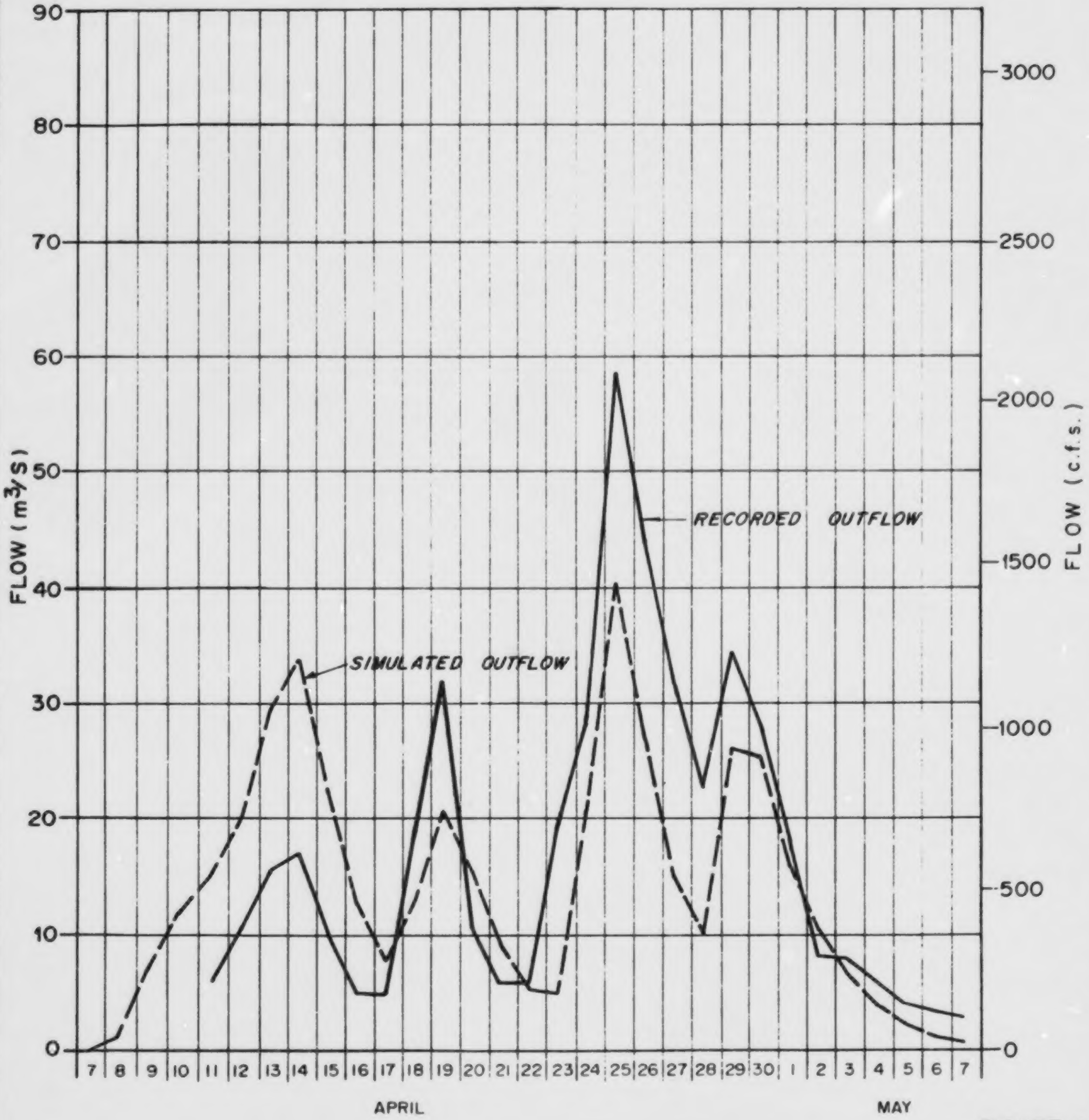


FIGURE 6

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

# WILLOW CREEK FLOWS
# 1970

**Manitoba**
**Natural Resources**
Water Resources

| | | |
|---|---|---|
| DRAWN<br>M. M. | SUBMITTED | SCALE<br>AS SHOWN |
| CHECKED | APPROVED | DATE<br>87 01 14 |
| PREPARED<br>G. MOHR | | SHEET<br>OF |
| | | FILE NO. |

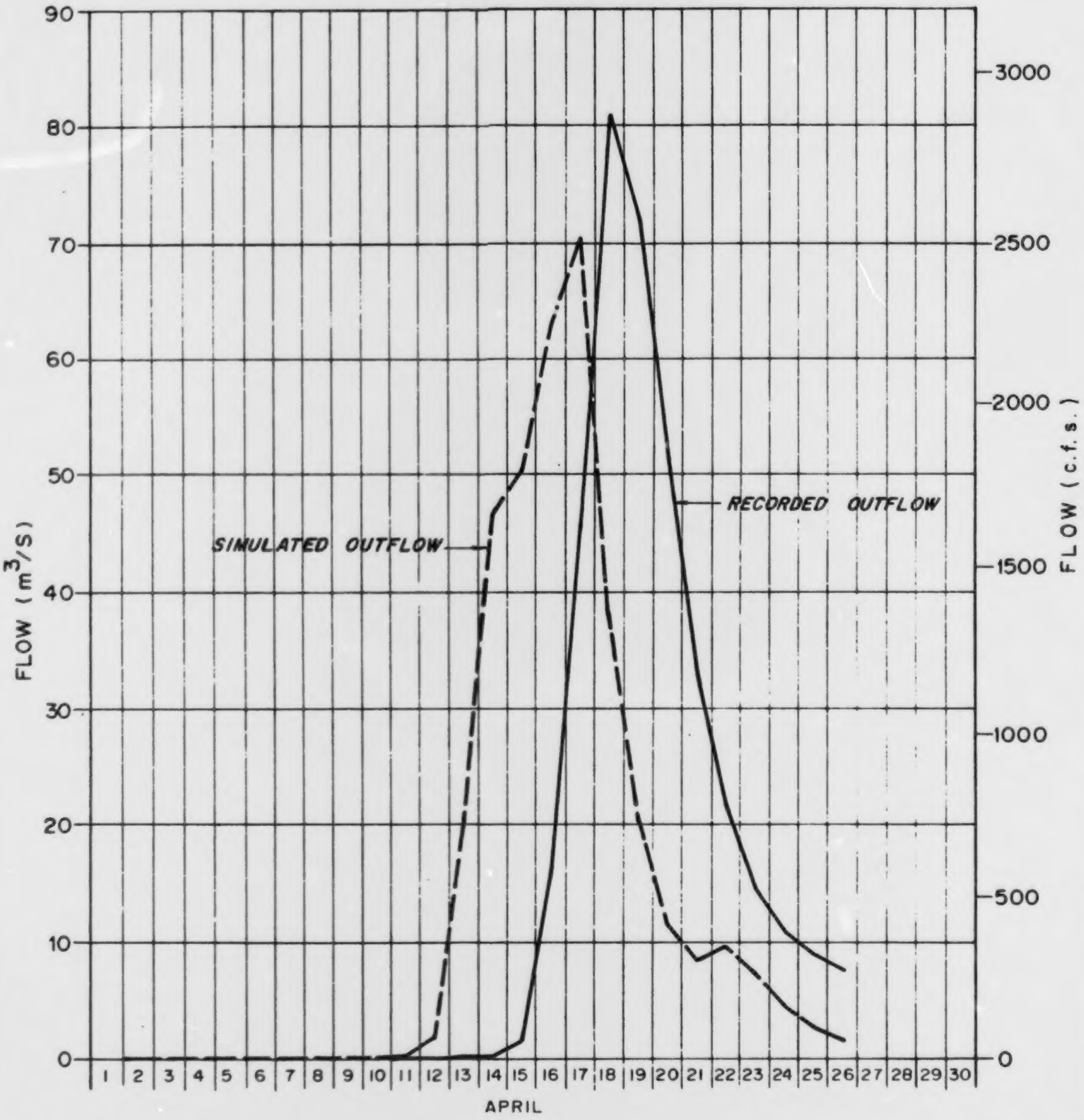


FIGURE 7

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

## WILLOW CREEK FLOWS 1974

| | | |
|---|---|---|
| DRAWN<br>M. M. | **Manitoba<br>Natural Resources**<br>Water Resources | |
| CHECKED | | |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED |

| | | | |
|---|---|---|---|
| SCALE<br>AS SHOWN | DATE<br>87 01 14 | SHEET<br>OF | FILE NO. |

1

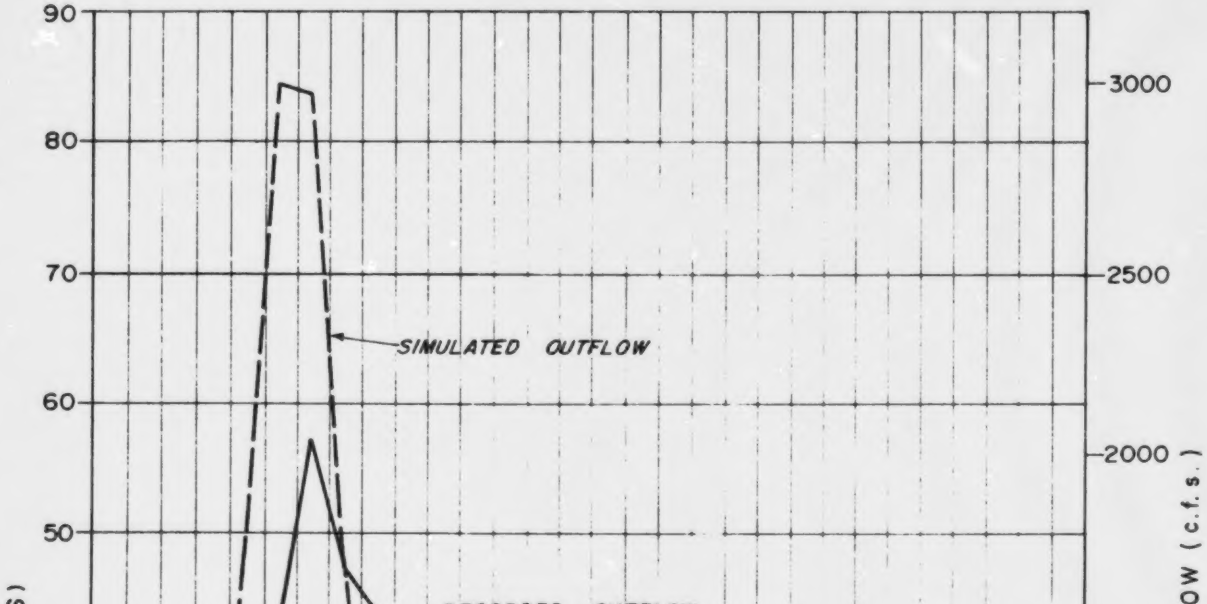


FIGURE 8

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

# WILLOW CREEK FLOWS
# 1976

| | | | | |
|---|---|---|---|---|
| DRAWN<br>M. M. | Manitoba<br>Natural Resources<br>Water Resources | | | |
| CHECKED | | | | |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED | | |
| SCALE<br>AS SHOWN | DATE<br>87 01 14 | SHEET<br>OF | FILE NO | |

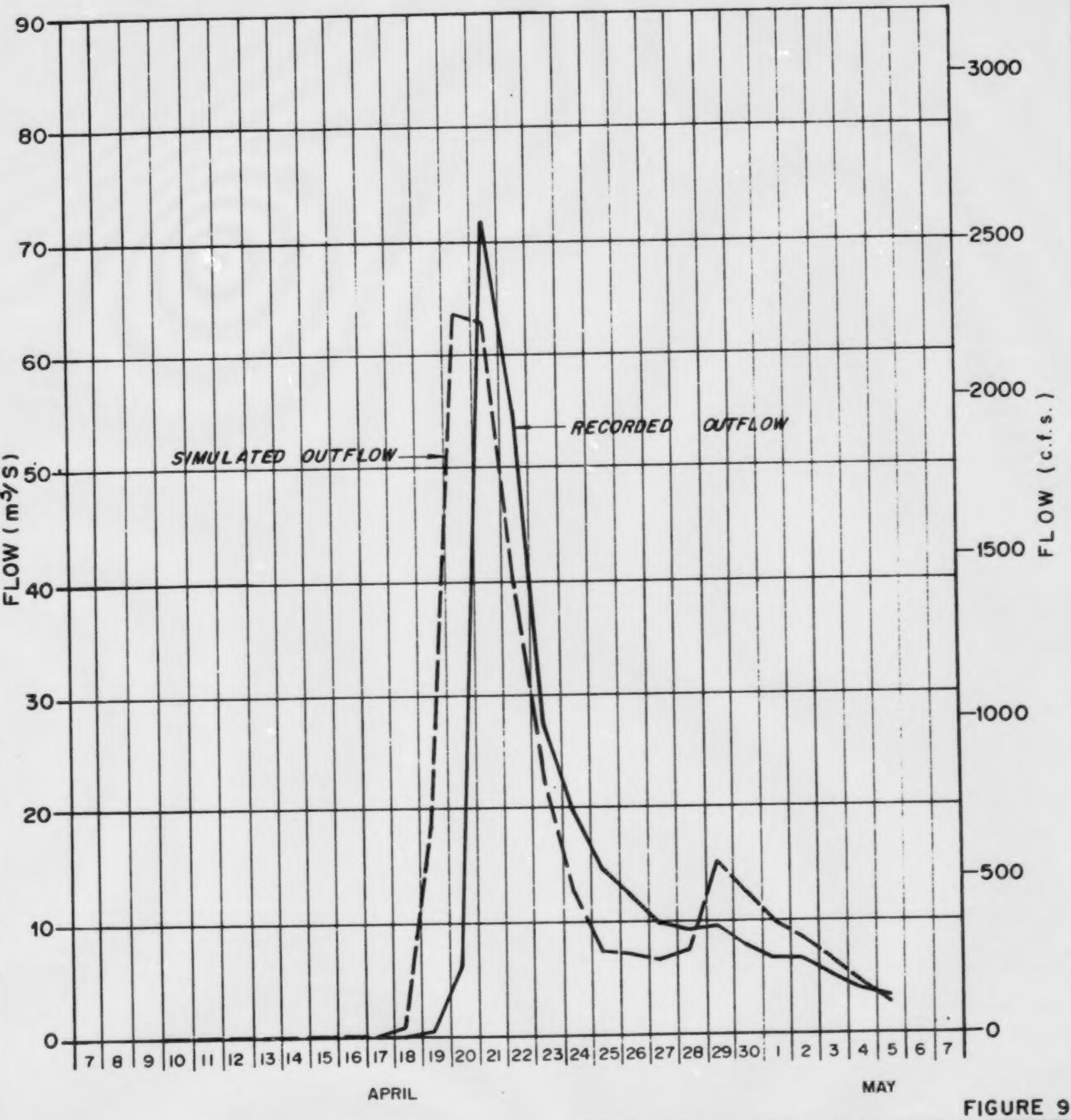


FIGURE 9

| | | |
|---|---|---|
| DRAWN<br>M. M. | **Manitoba Natural Resources**<br>Water Resources | CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM<br>FLOOD FORECASTING AGREEMENT- PHASE TWO<br>WILLOW CREEK FLOWS 1979 |
| CHECKED | | |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED |

| SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|
| AS SHOWN | 87 01 14 | OF | |

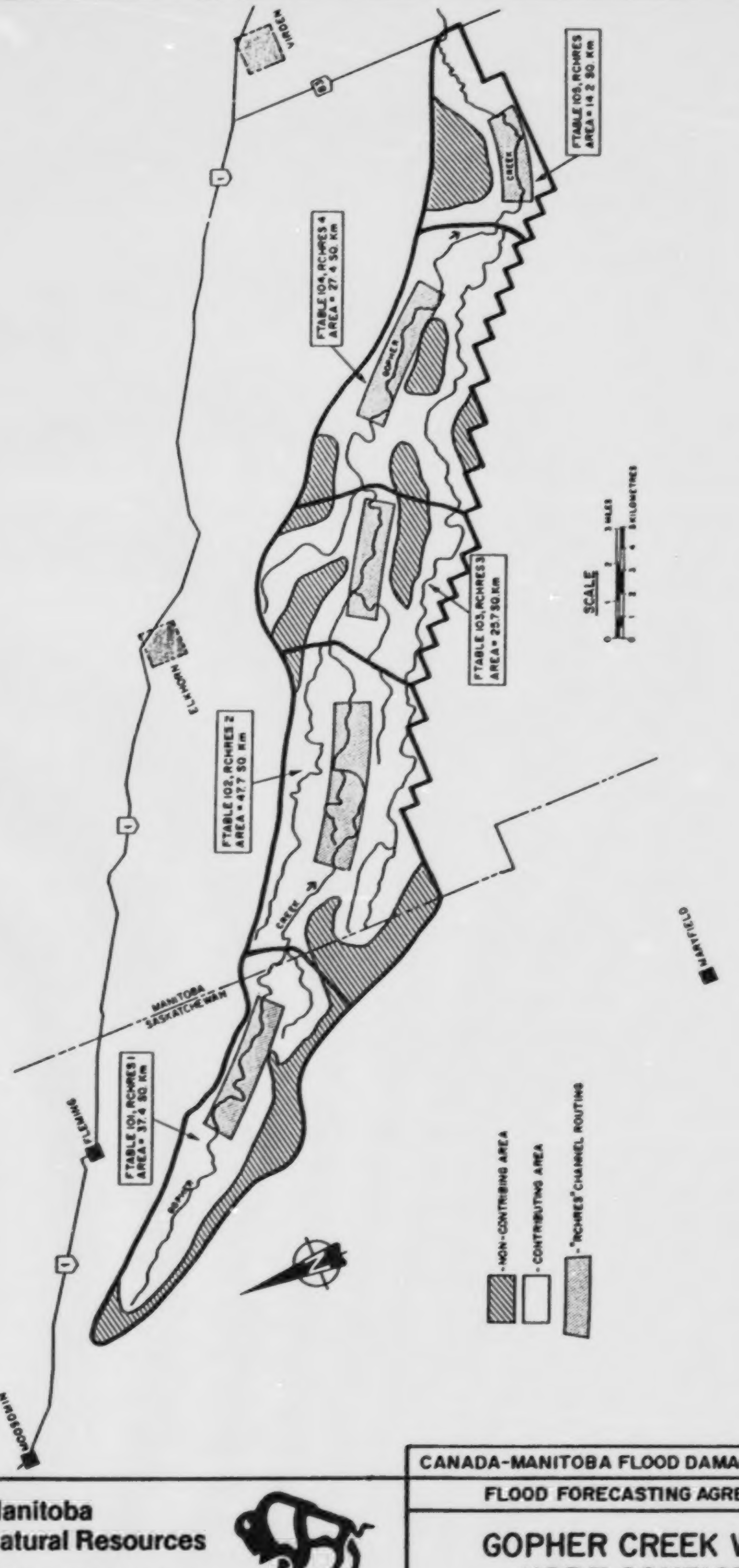

VIRDEN
83
1
ELKHORN
FLEMING
MOOSOMIN
MARYFIELD
MANITOBA
SASKATCHEWAN
GOPHER
CREEK
FTABLE 105, RCHRES
AREA = 14.2 SQ. Km
FTABLE 104, RCHRES 4
AREA = 27.4 SQ. Km
FTABLE 103, RCHRES 3
AREA = 25.7 SQ. Km
FTABLE 102, RCHRES 2
AREA = 47.7 SQ. Km
FTABLE 101, RCHRES 1
AREA = 37.4 SQ. Km
SCALE
MILES
KILOMETRES
N
▨ - NON-CONTRIBUTING AREA
☐ - CONTRIBUTING AREA
▩ - "RCHRES" CHANNEL ROUTING

FIGURE 10

| | | | | | |
|---|---|---|---|---|---|
| CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM | | | | | |
| FLOOD FORECASTING AGREEMENT-PHASE TWO | | | | | |
| DRAWN<br>J. MAMOTT | Manitoba<br>Natural Resources<br>Water Resources | | GOPHER CREEK WATERSHED<br>HSPF CONFIGERATION | | |
| CHECKED | | | | | |
| PREPARED<br>E. BLAIS | SUBMITTED | APPROVED | SCALE<br>AS SHOWN | DATE<br>87 01 19 | SHEET<br>OF | FILE NO. |

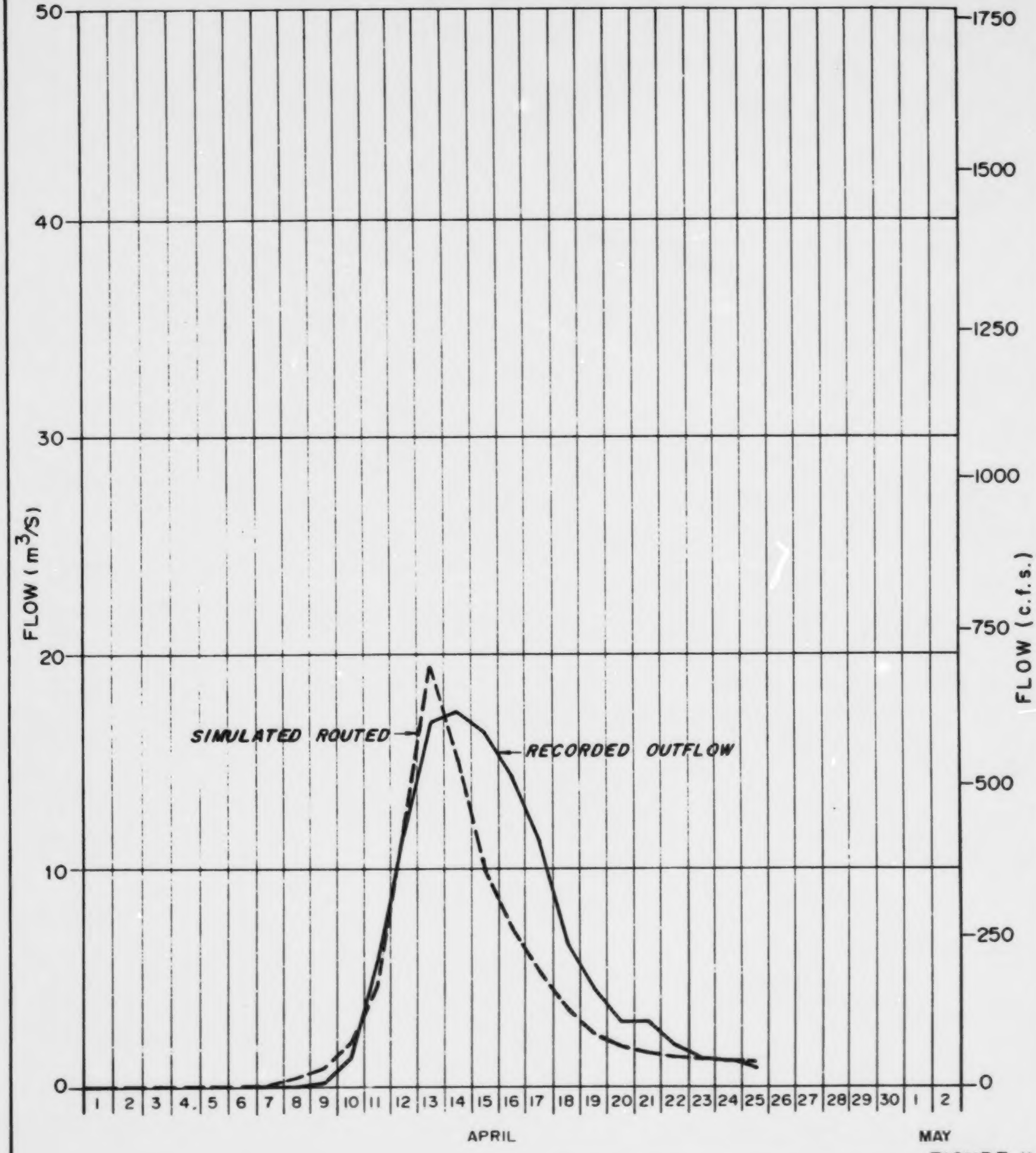


FIGURE II

| | | | |
|---|---|---|---|
| CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM | | | |
| FLOOD FORECASTING AGREEMENT-PHASE TWO | | | |
| GOPHER CREEK FLOWS<br>1969 | | | |
| SCALE AS SHOWN | DATE 87 01 12 | SHEET OF | FILE NO. |

Manitoba
Natural Resources
Water Resources

| | | |
|---|---|---|
| DRAWN M. M. | | |
| CHECKED | | |
| PREPARED G. MOHR | SUBMITTED | APPROVED |

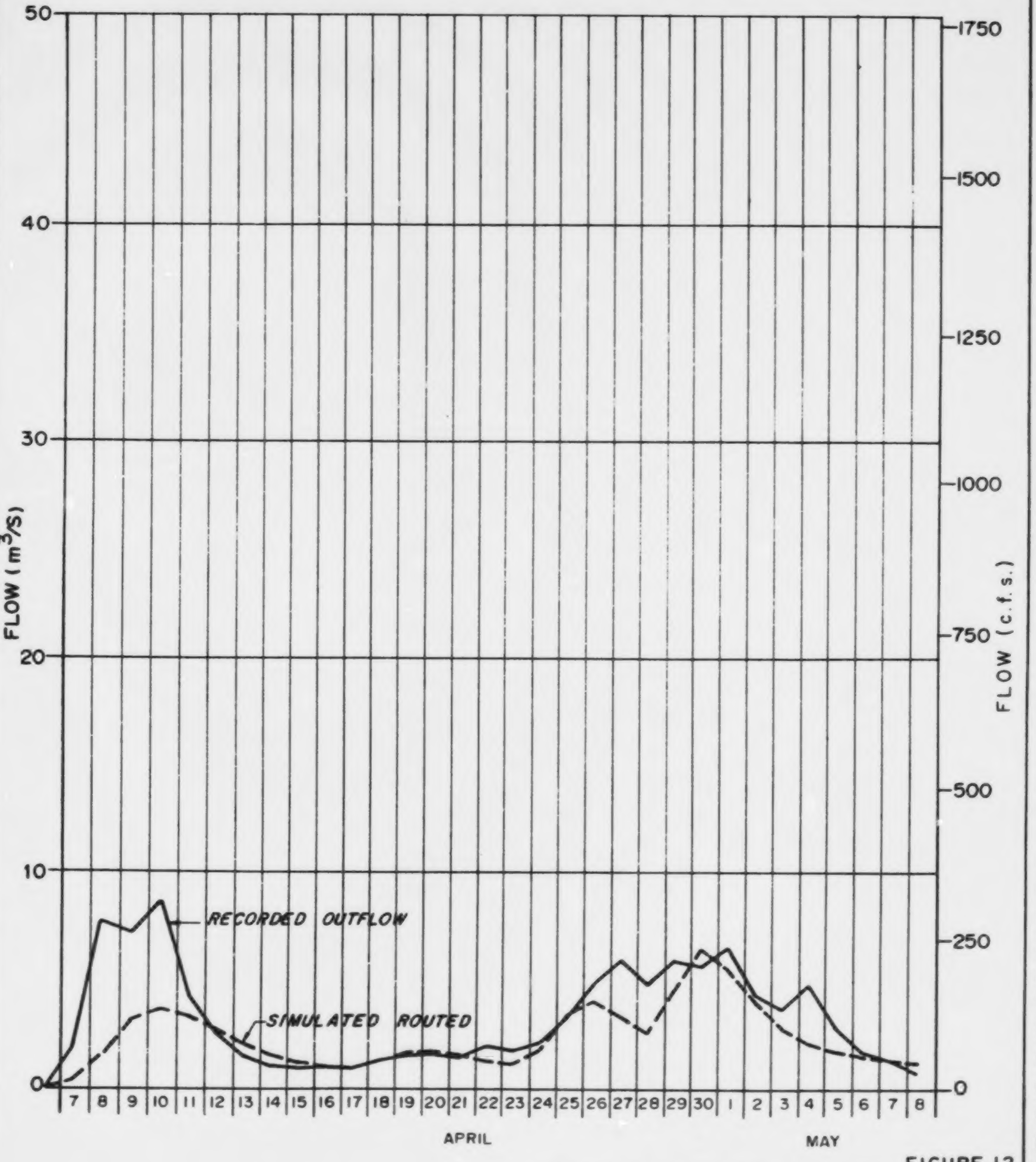


FIGURE 12

| | | |
|---|---|---|
| DRAWN<br>M. M. | **Manitoba**<br>**Natural Resources**<br>Water Resources | CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM<br>FLOOD FORECASTING AGREEMENT-PHASE TWO<br>GOPHER CREEK FLOWS<br>1970 |
| CHECKED | | |
| PREPARED<br>G. MOHR | SUBMITTED / APPROVED | SCALE AS SHOWN / DATE 87 01 12 / SHEET OF / FILE NO. |

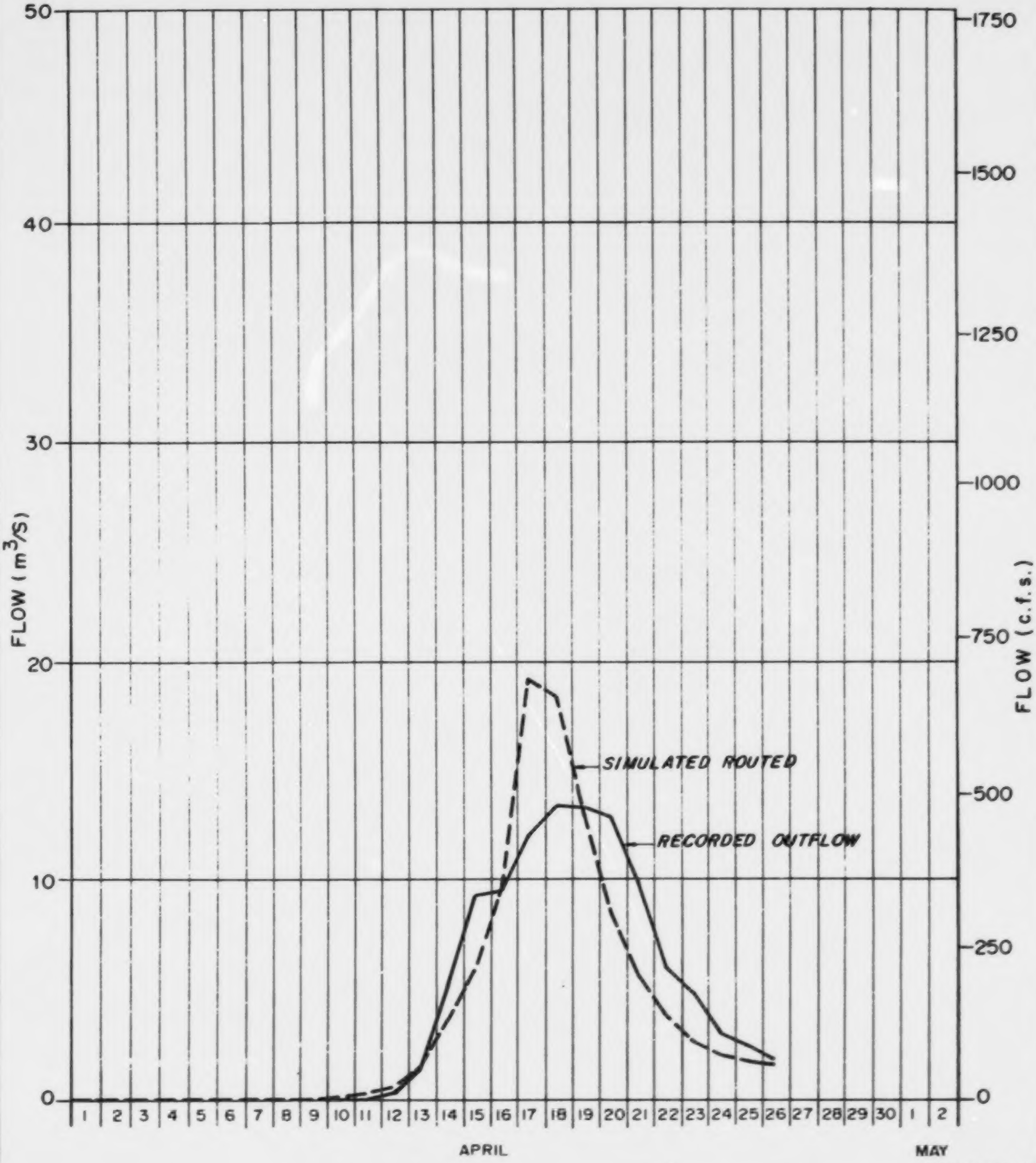


FIGURE 13

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

## GOPHER CREEK FLOWS
## 1974

| | | | | |
|---|---|---|---|---|
| DRAWN<br>M. M. | Manitoba<br>Natural Resources<br>Water Resources | | | |
| CHECKED | | | | |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED | | |
| SCALE<br>AS SHOWN | DATE<br>87 01 12 | SHEET<br>OF | FILE NO. | |

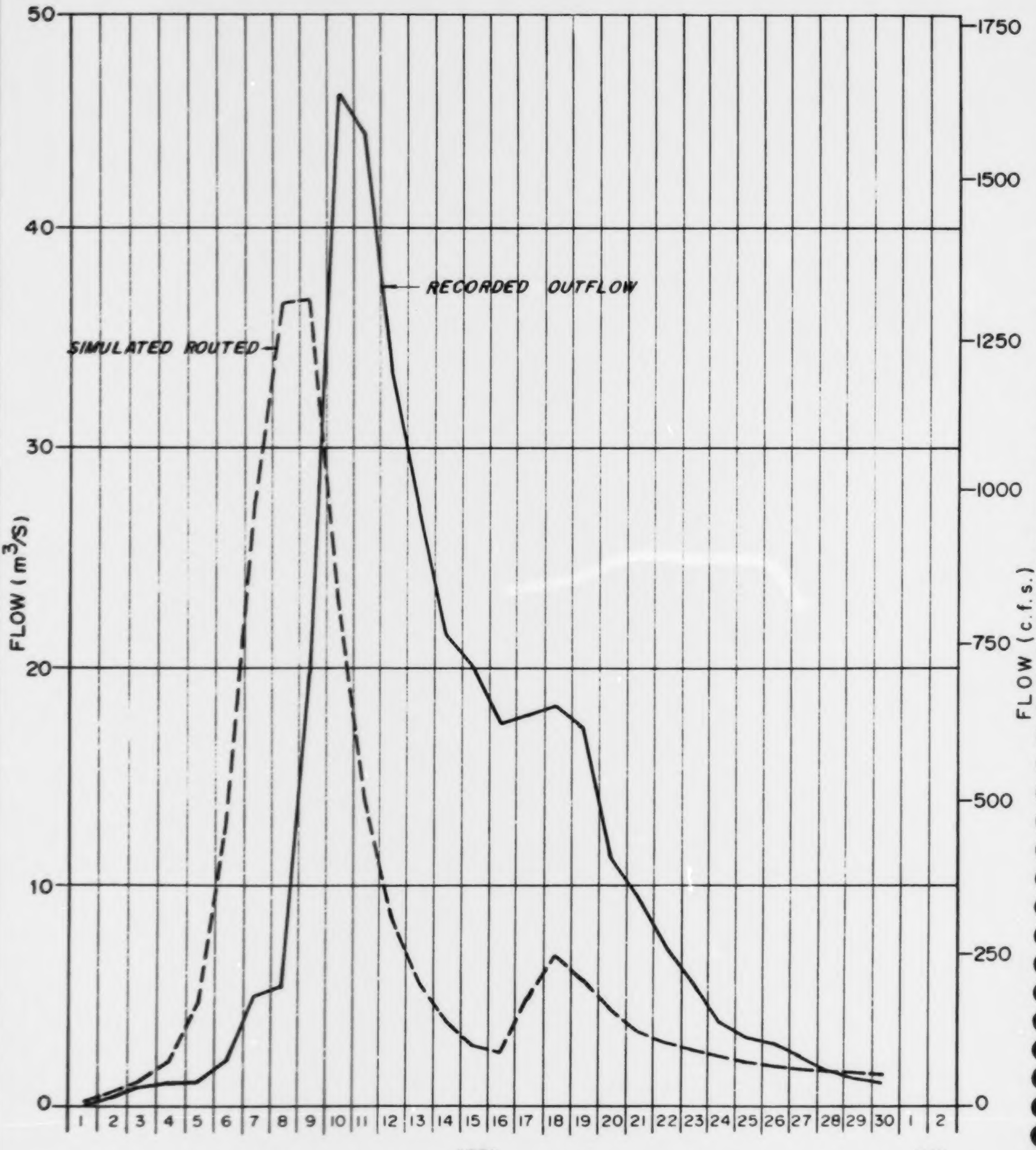


FIGURE 14

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

# GOPHER CREEK FLOWS 1976

**Manitoba Natural Resources**

Water Resources

DRAWN M. M.

CHECKED

PREPARED G. MOHR

SUBMITTED

APPROVED

SCALE AS SHOWN

DATE 87 01 12

SHEET OF

FILE NO.

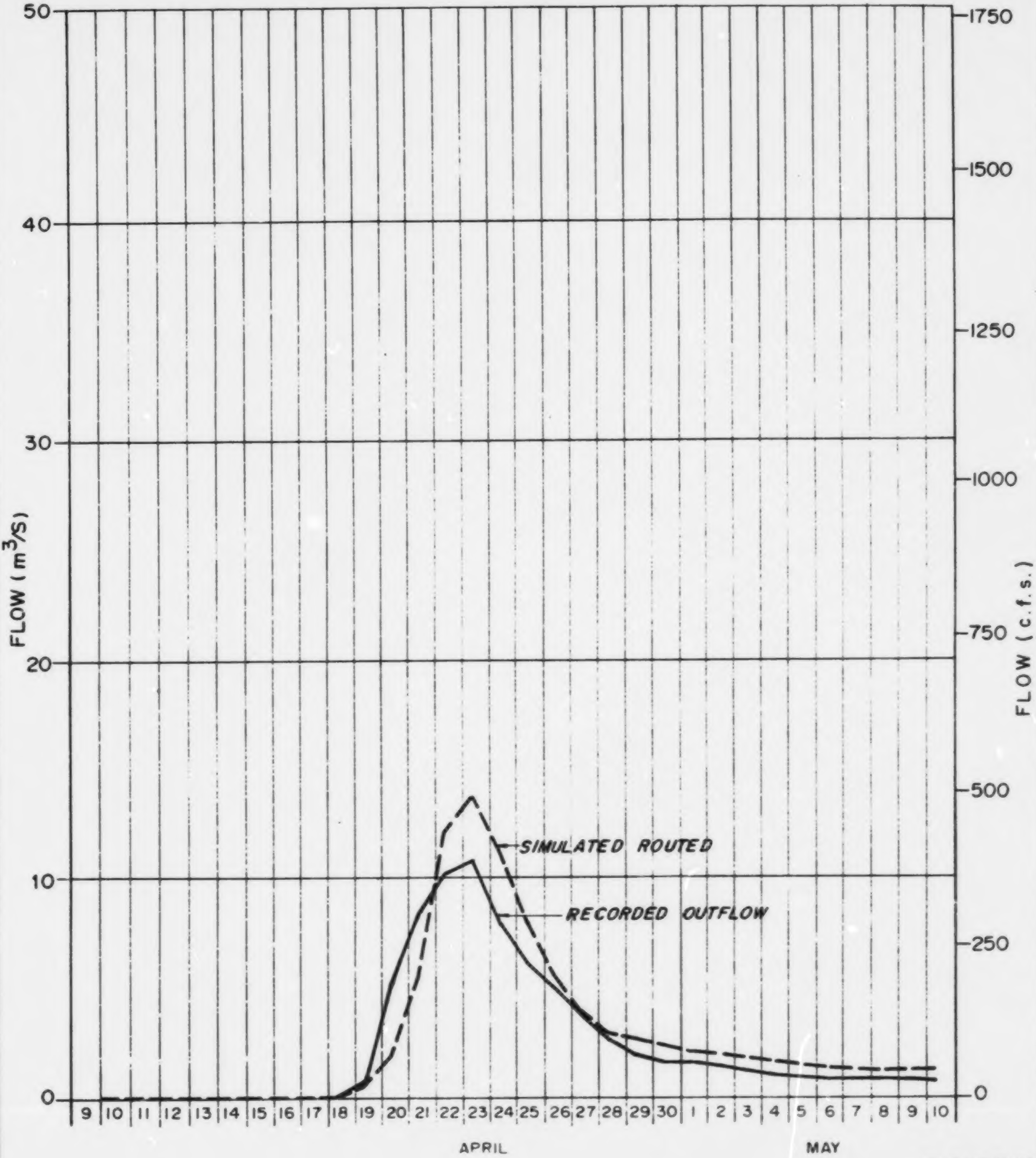


FIGURE 15

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

# GOPHER CREEK FLOWS
# 1979

DRAWN M. M.
CHECKED
PREPARED G. MOHR

**Manitoba**
**Natural Resources**
Water Resources

SUBMITTED
APPROVED

SCALE AS SHOWN
DATE 87 01 12
SHEET OF
FILE NO

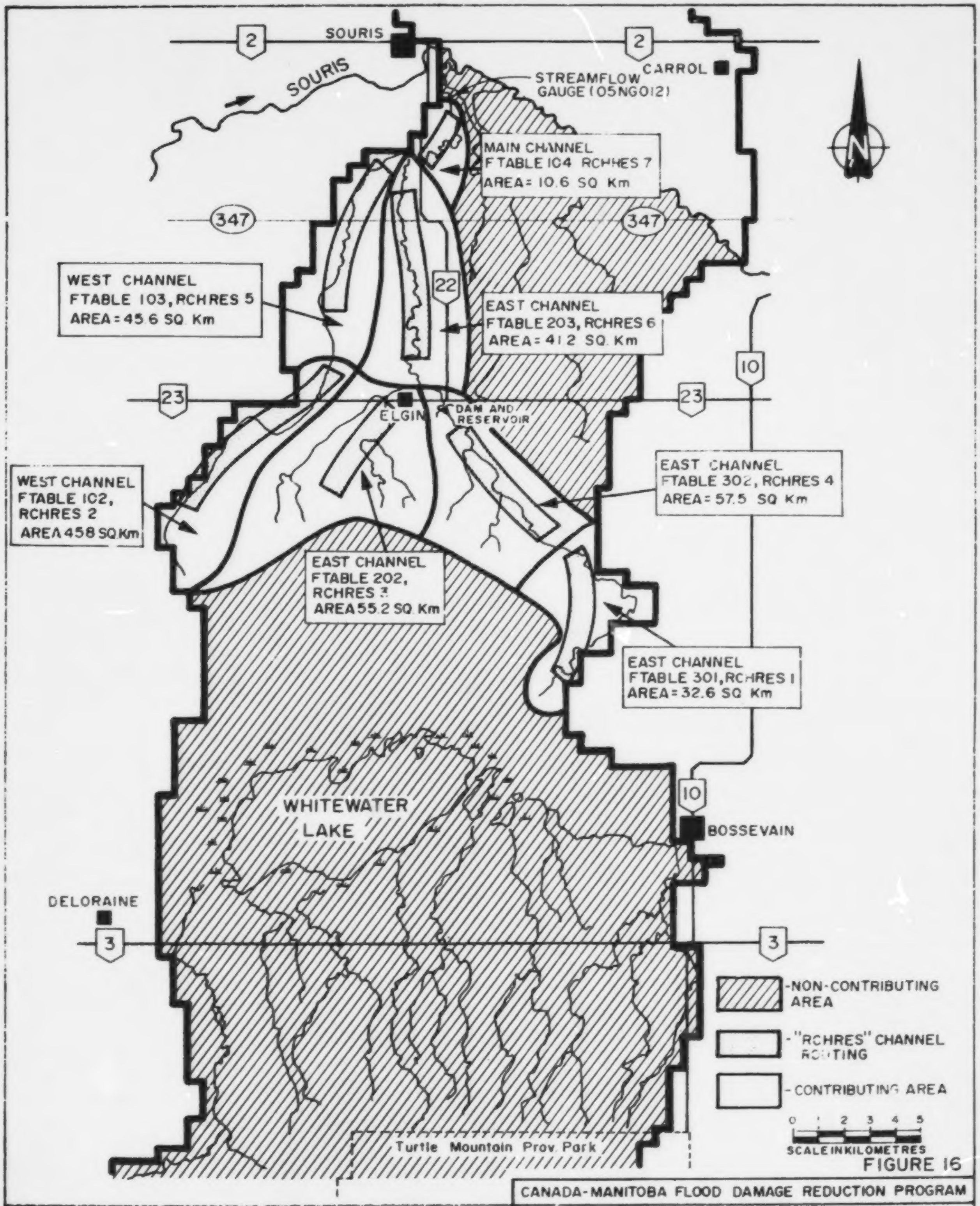

SOURIS
2
2
CARROL
SOURIS
STREAMFLOW
GAUGE (05NG012)
MAIN CHANNEL
FTABLE 104 RCHRES 7
AREA = 10.6 SQ Km
347
347
WEST CHANNEL
FTABLE 103, RCHRES 5
AREA = 45.6 SQ Km
22
EAST CHANNEL
FTABLE 203, RCHRES 6
AREA = 41.2 SQ Km
10
23
23
ELGIN
DAM AND
RESERVOIR
EAST CHANNEL
FTABLE 302, RCHRES 4
AREA = 57.5 SQ Km
WEST CHANNEL
FTABLE 102,
RCHRES 2
AREA 45.8 SQ Km
EAST CHANNEL
FTABLE 202,
RCHRES 3
AREA 55.2 SQ Km
EAST CHANNEL
FTABLE 301, RCHRES 1
AREA = 32.6 SQ Km
WHITEWATER
LAKE
10
BOSSEVAIN
DELORAINE
3
3
-NON-CONTRIBUTING
AREA
-"RCHRES" CHANNEL
ROUTING
-CONTRIBUTING AREA
0 1 2 3 4 5
SCALE IN KILOMETRES
Turtle Mountain Prov. Park
N

FIGURE 16

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT - PHASE TWO

## ELGIN CREEK WATERSHED
## HSPF CONFIGURATION

Manitoba
Natural Resources
Water Resources

| | | | |
|---|---|---|---|
| DRAWN<br>J. MAMOTT | CHECKED | PREPARED<br>G. MOHR | SUBMITTED |
| APPROVED | SCALE<br>AS SHOWN | DATE<br>86 01 05 | SHEET<br>OF |
| FILE NO | | | |

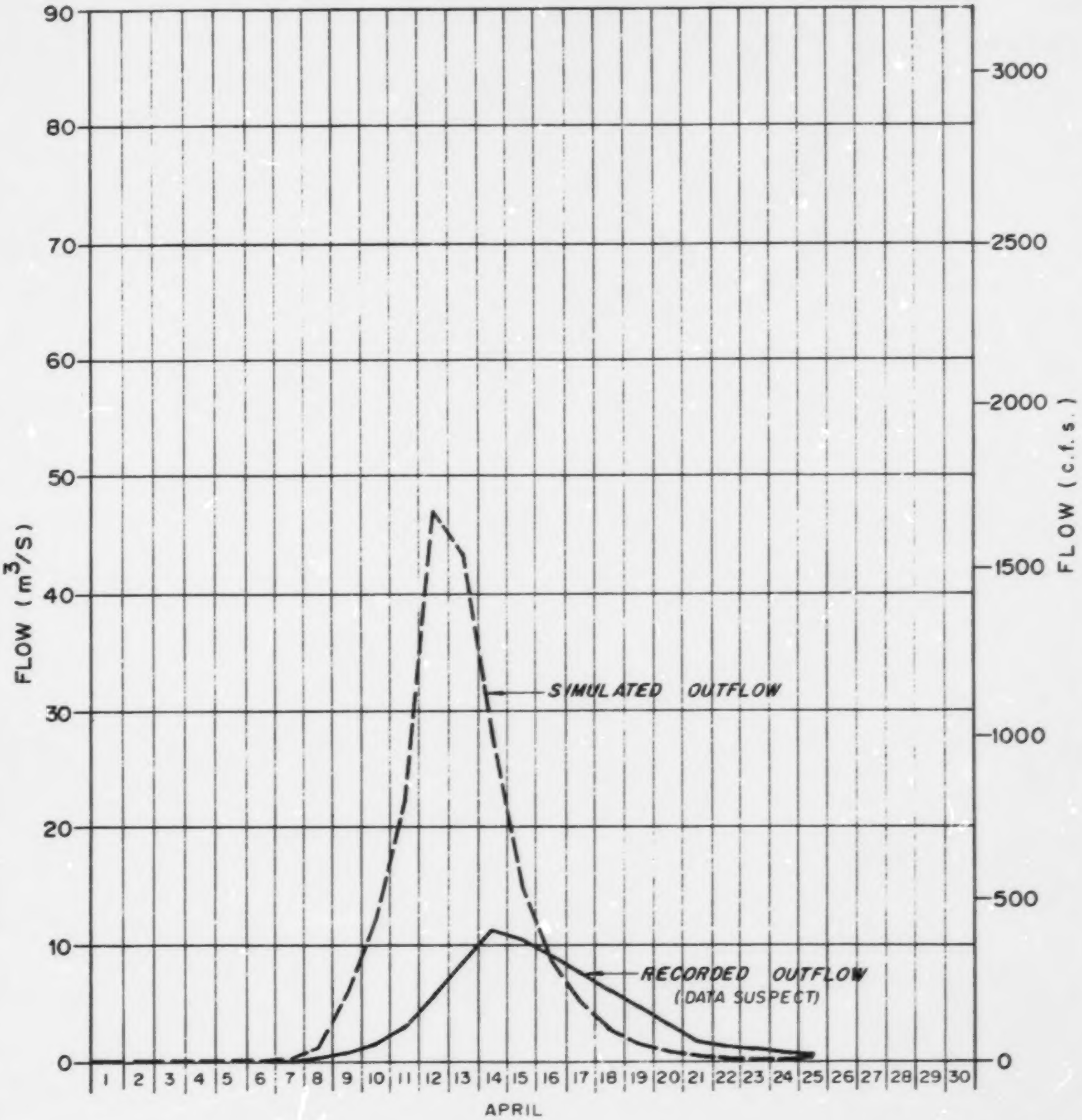


FIGURE 17

| | | | |
|---|---|---|---|
| DRAWN<br>M. M. | Manitoba<br>Natural Resources<br>Water Resources | | CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM<br>FLOOD FORECASTING AGREEMENT- PHASE TWO |
| CHECKED | | | ELGIN CREEK FLOWS<br>1969 |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED | SCALE AS SHOWN &#124; DATE 87 01 12 &#124; SHEET OF &#124; FILE NO |

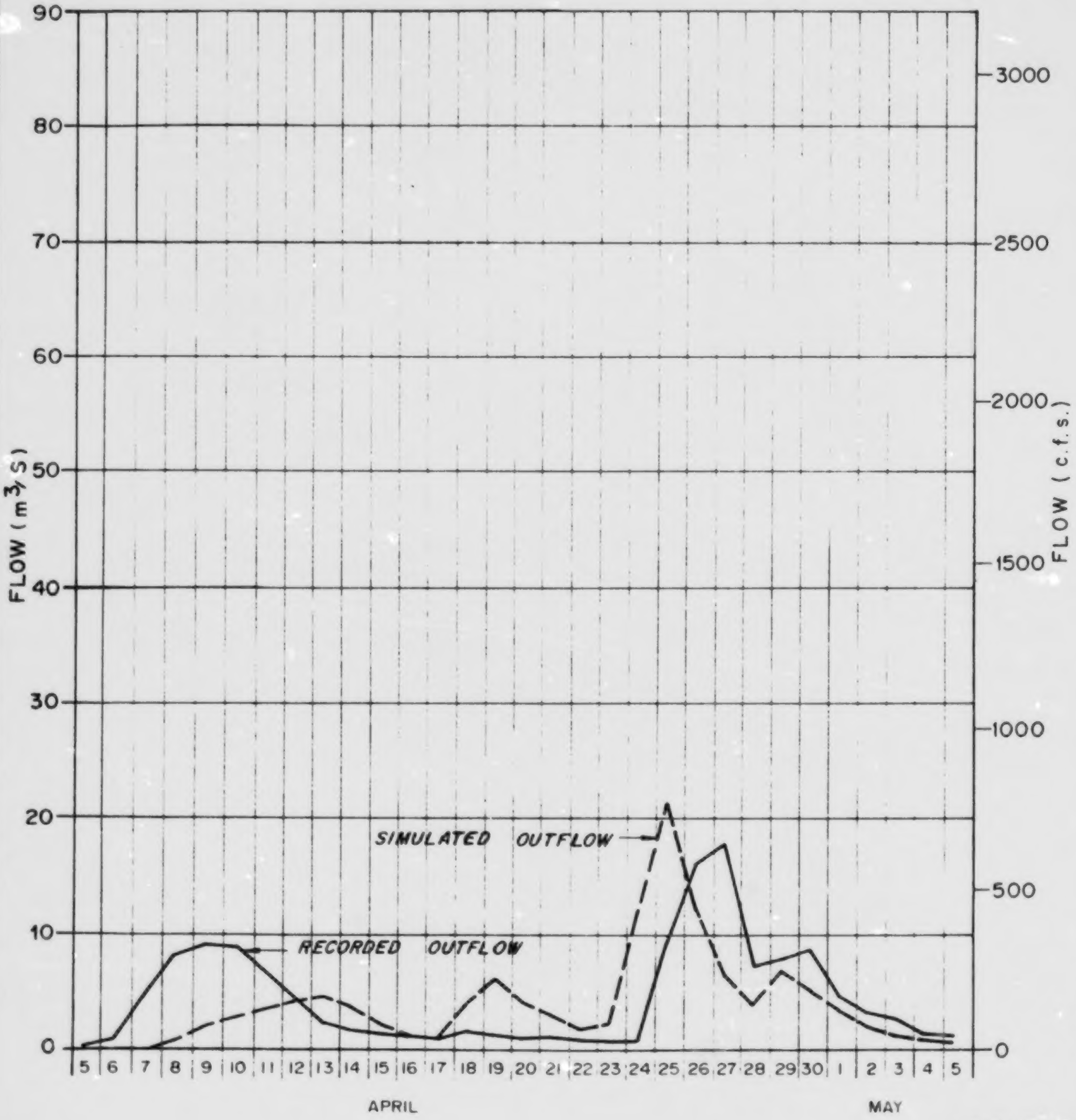


FIGURE 18

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

# ELGIN CREEK FLOWS 1970

| | | | |
|---|---|---|---|
| DRAWN<br>M. M. | Manitoba<br>Natural Resources<br>Water Resources | | |
| CHECKED | | | |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED | |

| SCALE | DATE | SHEET | FILE NO |
|---|---|---|---|
| AS SHOWN | 87 01 12 | OF | |

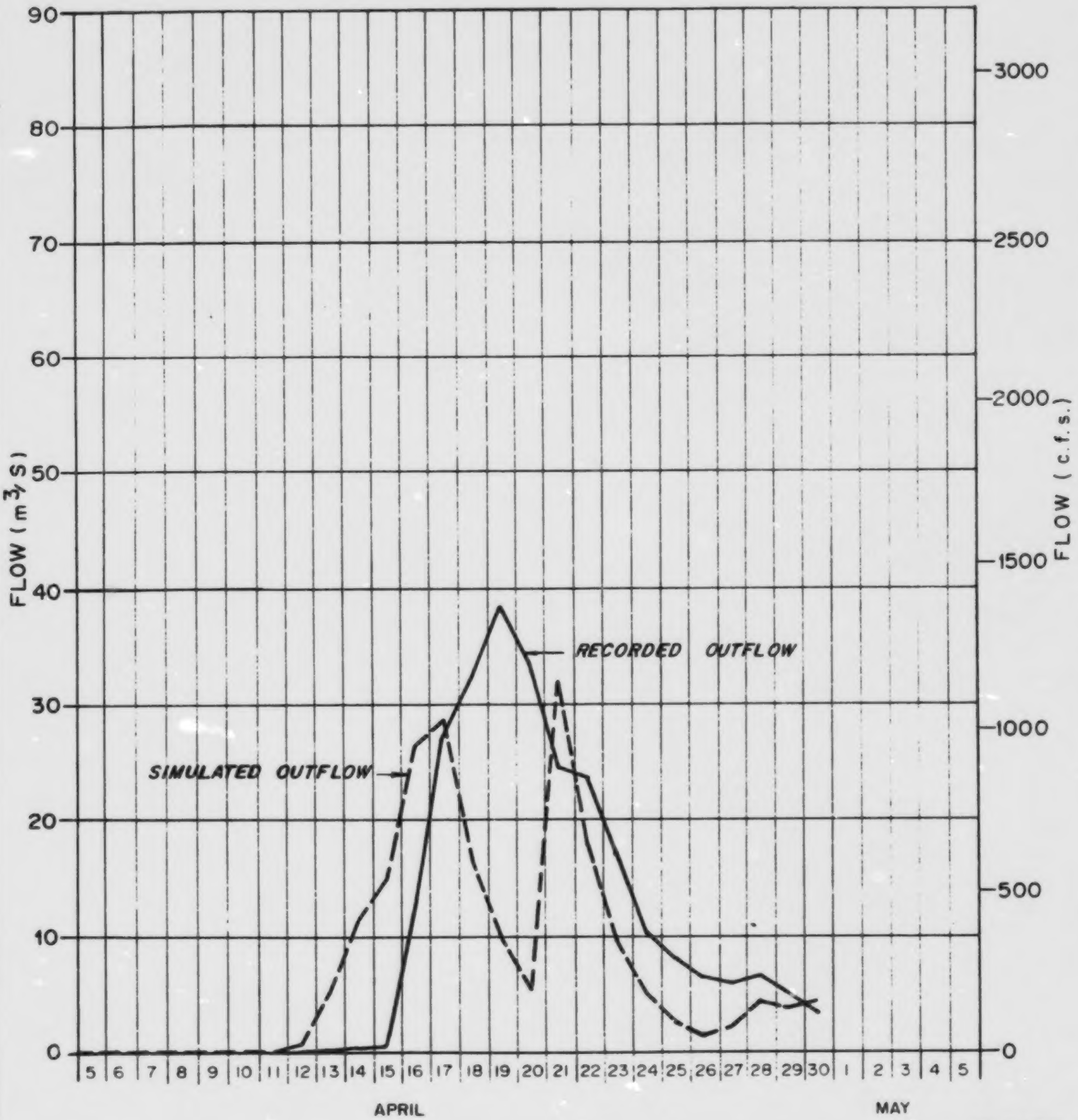


FIGURE 19

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

# ELGIN CREEK FLOWS
# 1974

**Manitoba**
**Natural Resources**
Water Resources

DRAWN M. M.
CHECKED
PREPARED G. MOHR
SUBMITTED
APPROVED
SCALE AS SHOWN
DATE 87 01 12
SHEET OF
FILE NO

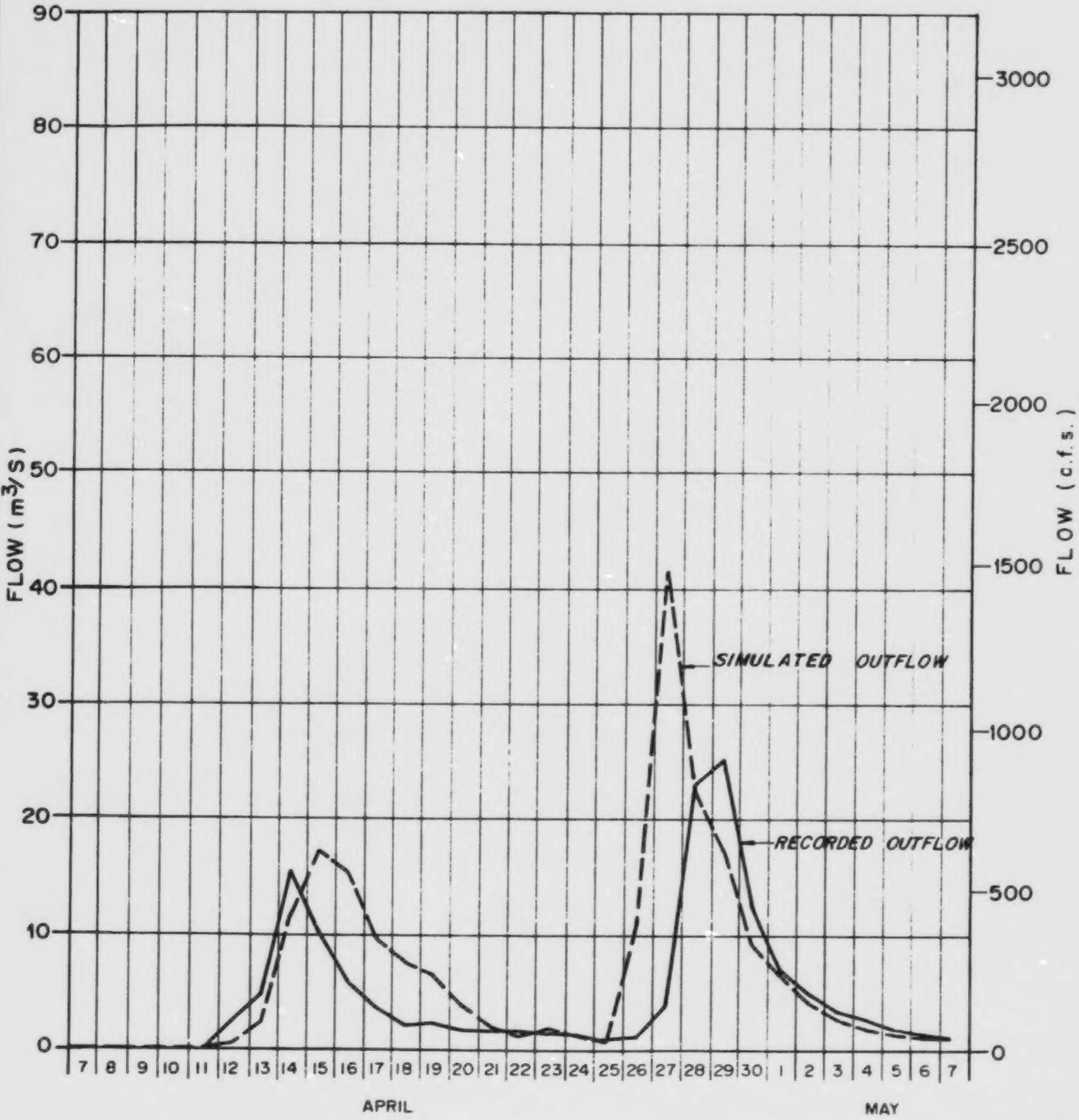


FIGURE 20

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

## ELGIN CREEK FLOWS
## 1975

**Manitoba**
**Natural Resources**
Water Resources

| | |
|---|---|
| DRAWN | M. M. |
| CHECKED | |
| PREPARED | G. MOHR |
| SUBMITTED | |
| APPROVED | |
| SCALE | AS SHOWN |
| DATE | 87 01 13 |
| SHEET | OF |
| FILE NO | |

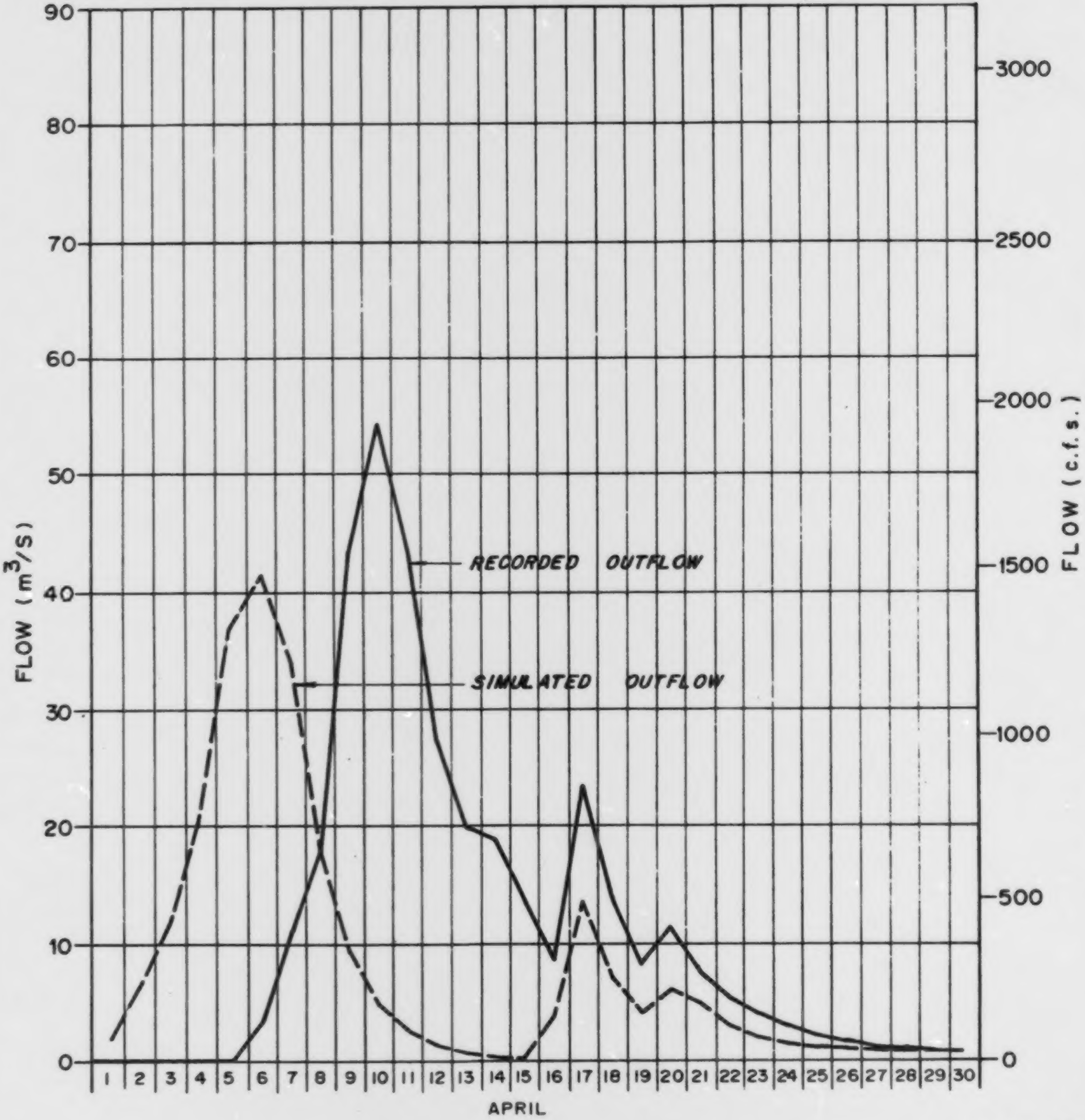


FIGURE 21

|  |  |
|---|---|
| DRAWN<br>M. M. |  |
| CHECKED |  |
| PREPARED<br>G. MOHR | SUBMITTED |

**Manitoba Natural Resources**

Water Resources

APPROVED

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

## ELGIN CREEK FLOWS
## 1976

| SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|
| AS SHOWN | 87 01 13 | OF |  |

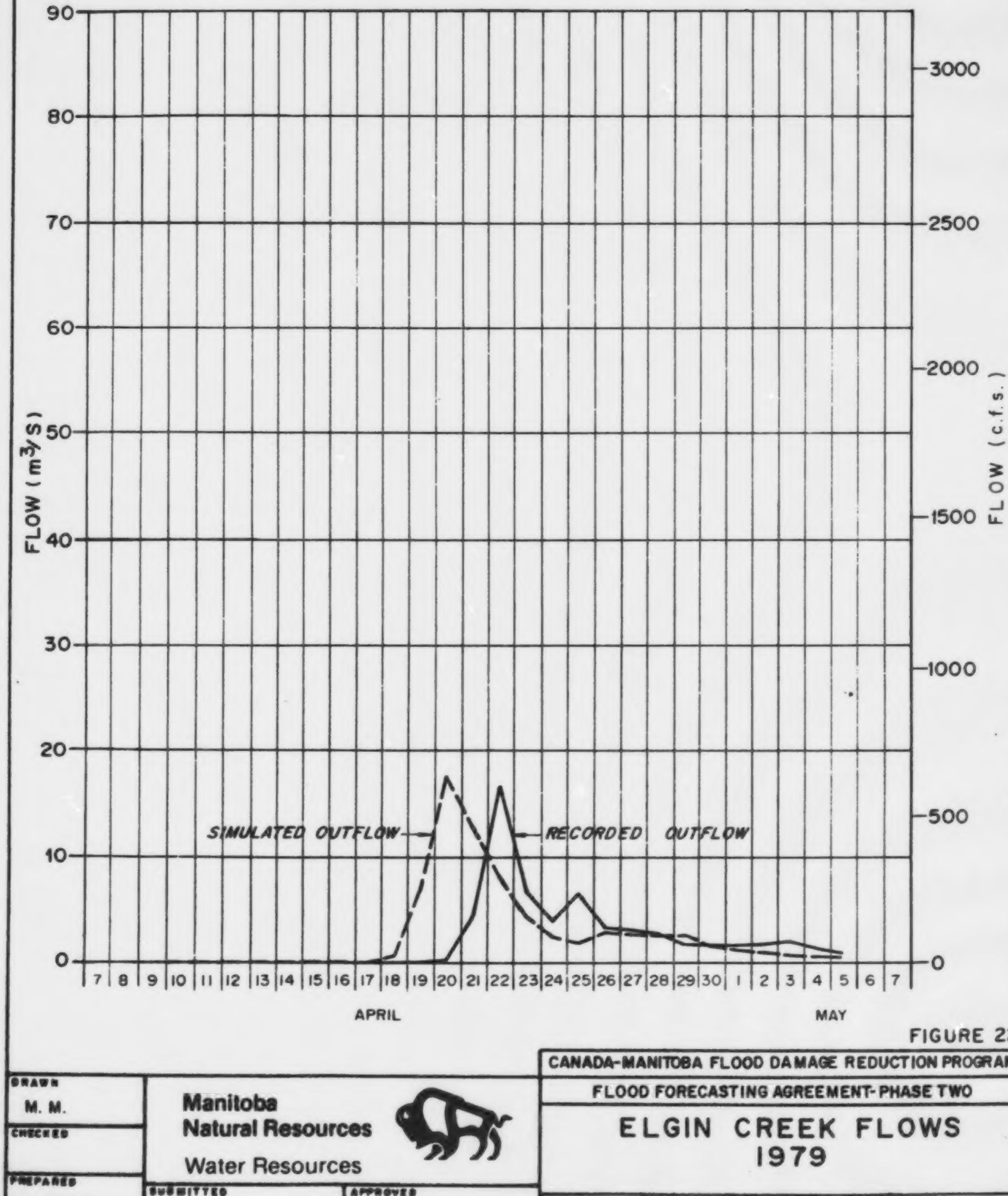


FIGURE 22

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

## ELGIN CREEK FLOWS
## 1979

| | | | | |
|---|---|---|---|---|
| DRAWN<br>M. M. | Manitoba<br>Natural Resources<br>Water Resources | | | |
| CHECKED | | | | |
| PREPARED<br>G. MOHR | SUBMITTED | APPROVED | | |
| SCALE<br>AS SHOWN | DATE<br>87 01 13 | SHEET<br>OF | FILE NO. | |

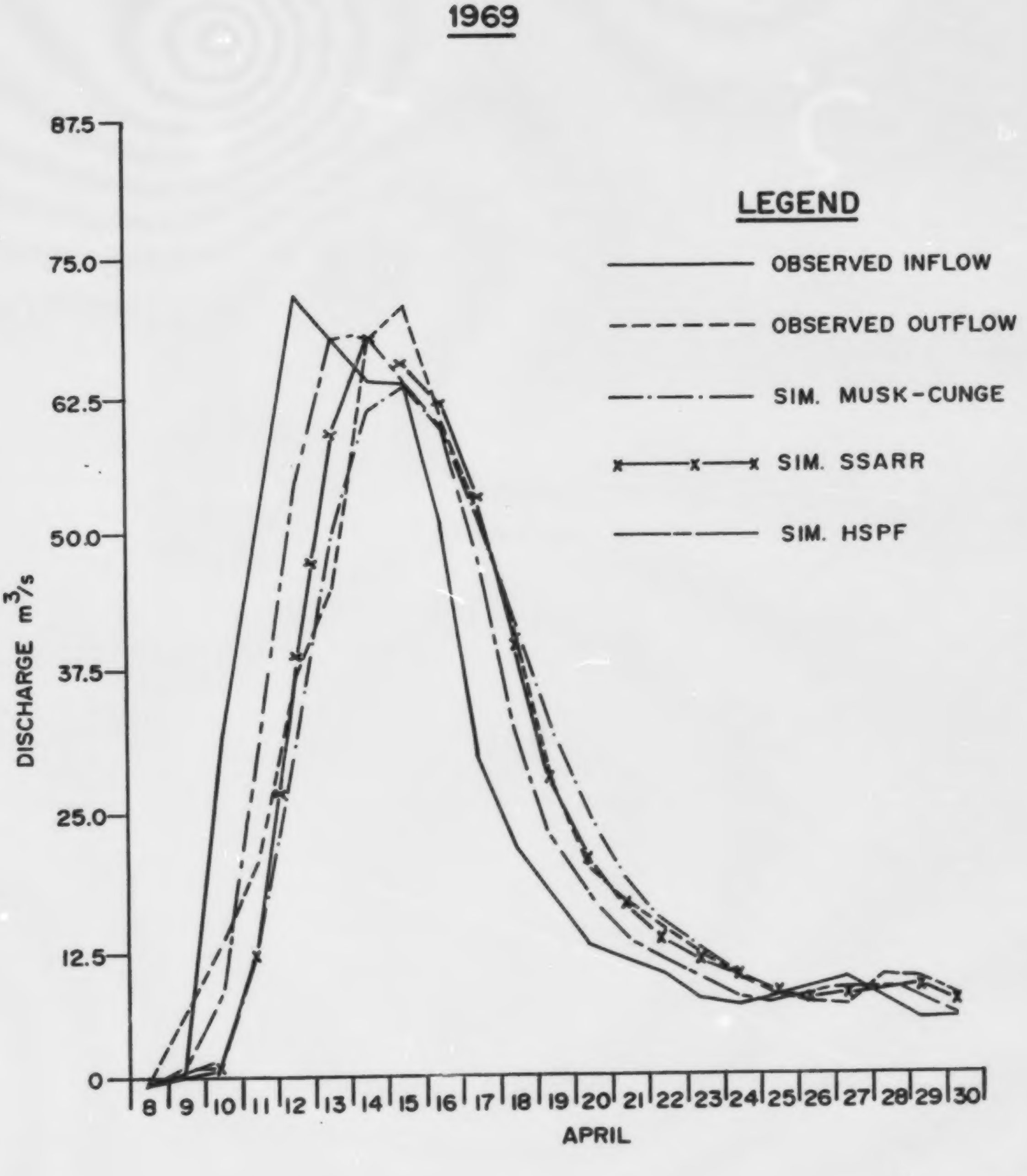


FIGURE 23

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT- PHASE TWO

TRIAL ROUTINGS

Manitoba
Natural Resources
Water Resources

DRAWN J. MAMOTT

CHECKED

PREPARED E. BLAIS

SUBMITTED

APPROVED

SCALE AS SHOWN

DATE 88 05 10

SHEET OF

FILE NO.

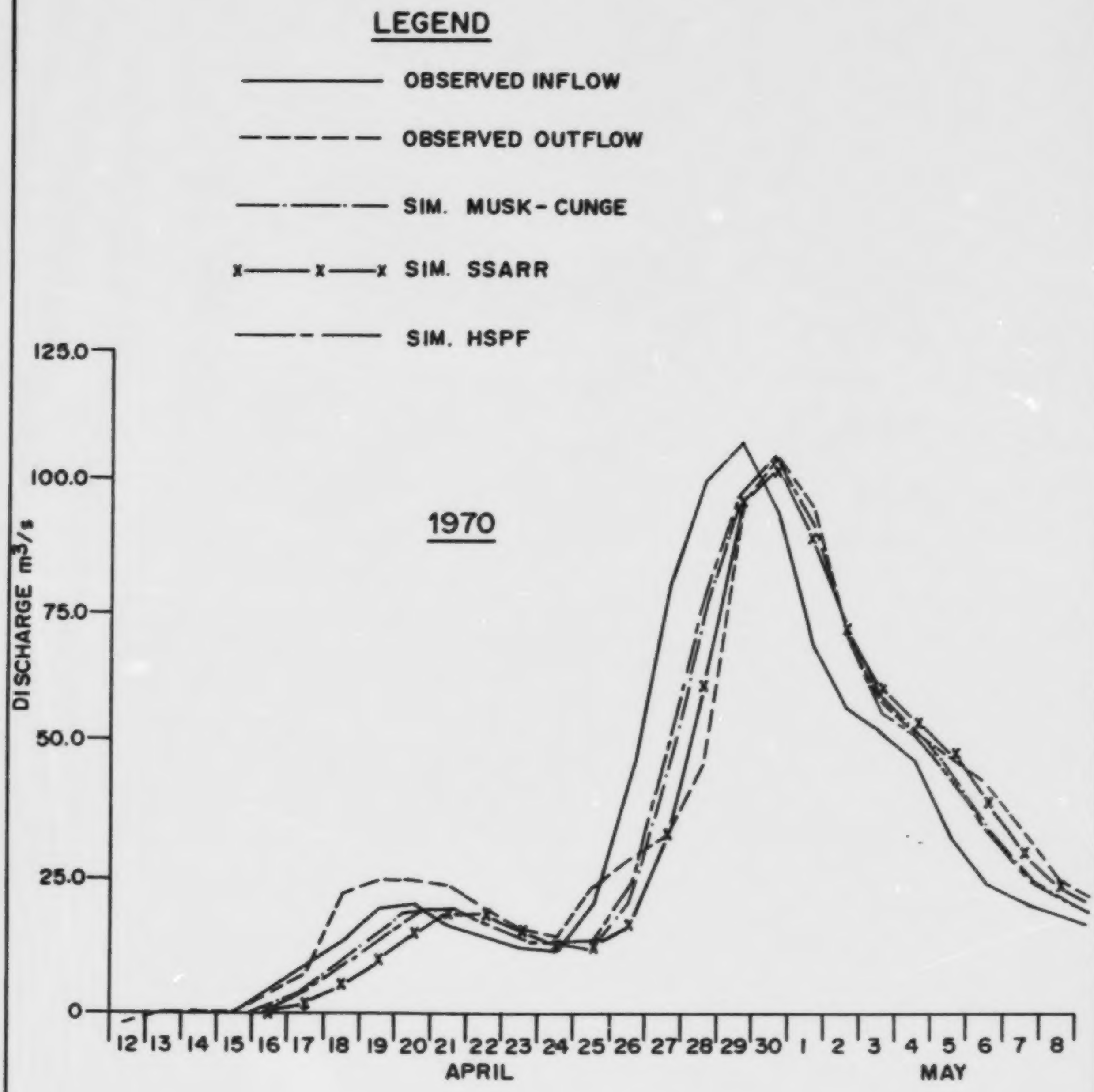


FIGURE 24

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

## TRIAL ROUTINGS

**Manitoba**
**Natural Resources**
Water Resources

DRAWN J. MAMOTT
CHECKED
PREPARED E. BLAIS
SUBMITTED
APPROVED
SCALE AS SHOWN
DATE 86 05 10
SHEET OF
FILE NO.

## 1971

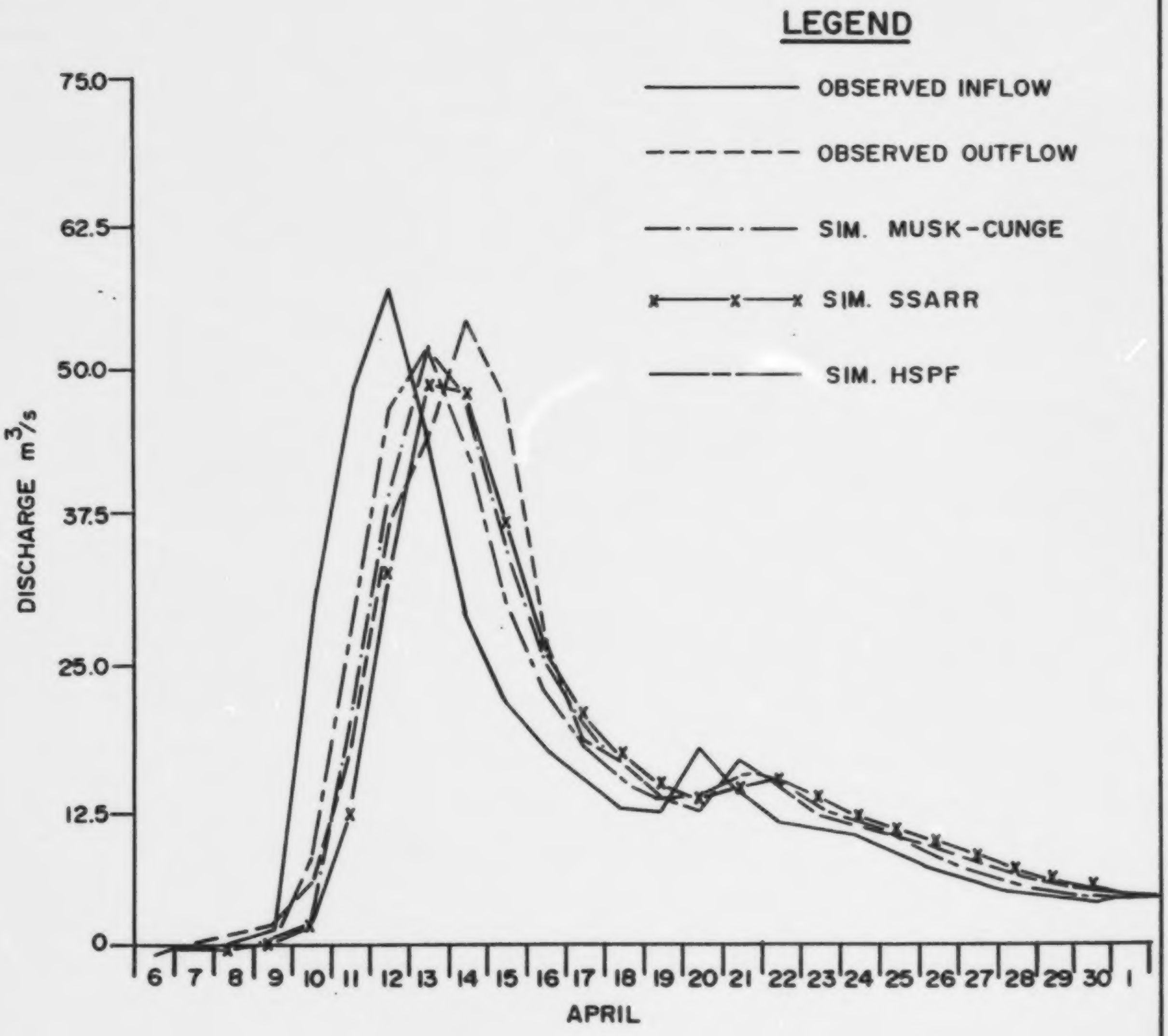


FIGURE 25

| | | |
|---|---|---|
| DRAWN<br>J. MAMOTT | Manitoba<br>Natural Resources<br>Water Resources | CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM<br>FLOOD FORECASTING AGREEMENT- PHASE TWO |
| CHECKED | | TRIAL ROUTINGS |
| PREPARED<br>E. BLAIS | SUBMITTED | APPROVED |

| SCALE | DATE | SHEET | FILE NO. |
|---|---|---|---|
| AS SHOWN | 88 05 10 | OF | |

## 1974

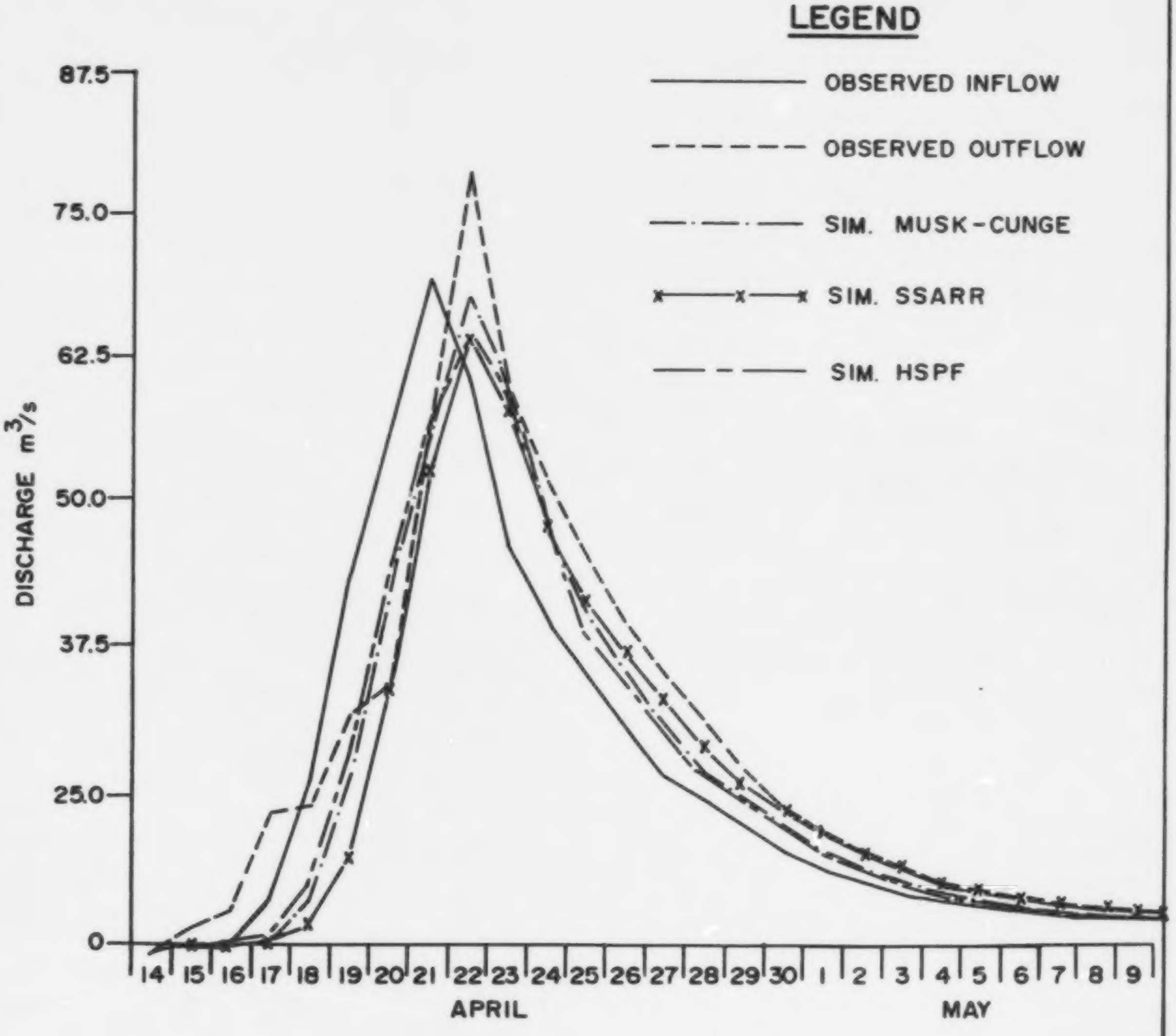


FIGURE 26

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

### TRIAL ROUTINGS

Manitoba
Natural Resources
Water Resources

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CHECKED
PREPARED E. BLAIS
SUBMITTED
APPROVED
SCALE AS SHOWN
DATE 88 05 10
SHEET OF
FILE NO.

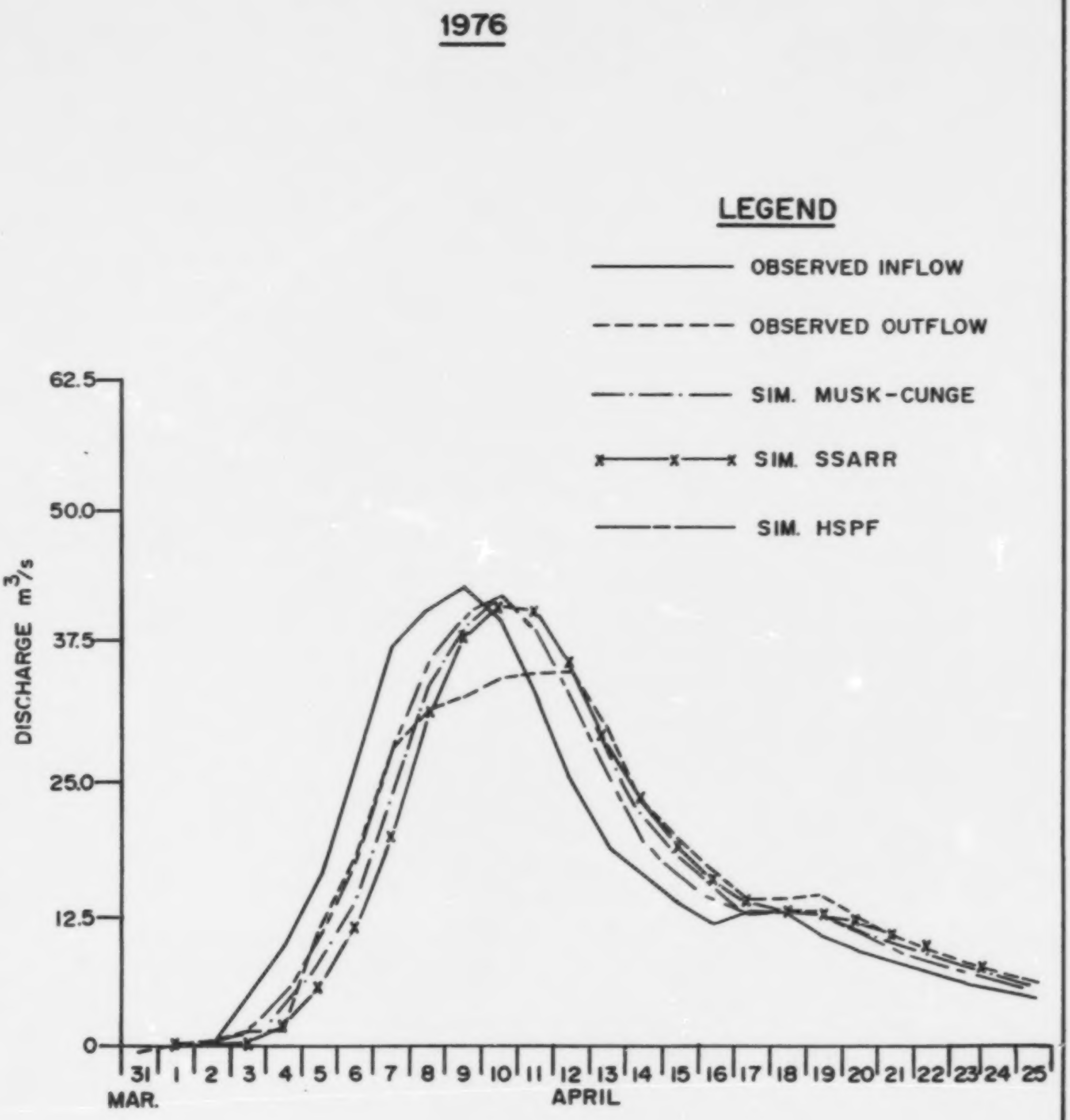

FIGURE 27

| | | | |
|---|---|---|---|
| CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM | | | |
| FLOOD FORECASTING AGREEMENT - PHASE TWO | | | |
| TRIAL ROUTINGS | | | |
| SCALE<br>AS SHOWN | DATE<br>88 05 10 | SHEET<br>OF | FILE NO. |

DRAWN J. MAMOTT

CHECKED

PREPARED E. BLAIS

**Manitoba Natural Resources**

Water Resources

SUBMITTED

APPROVED

## 1979

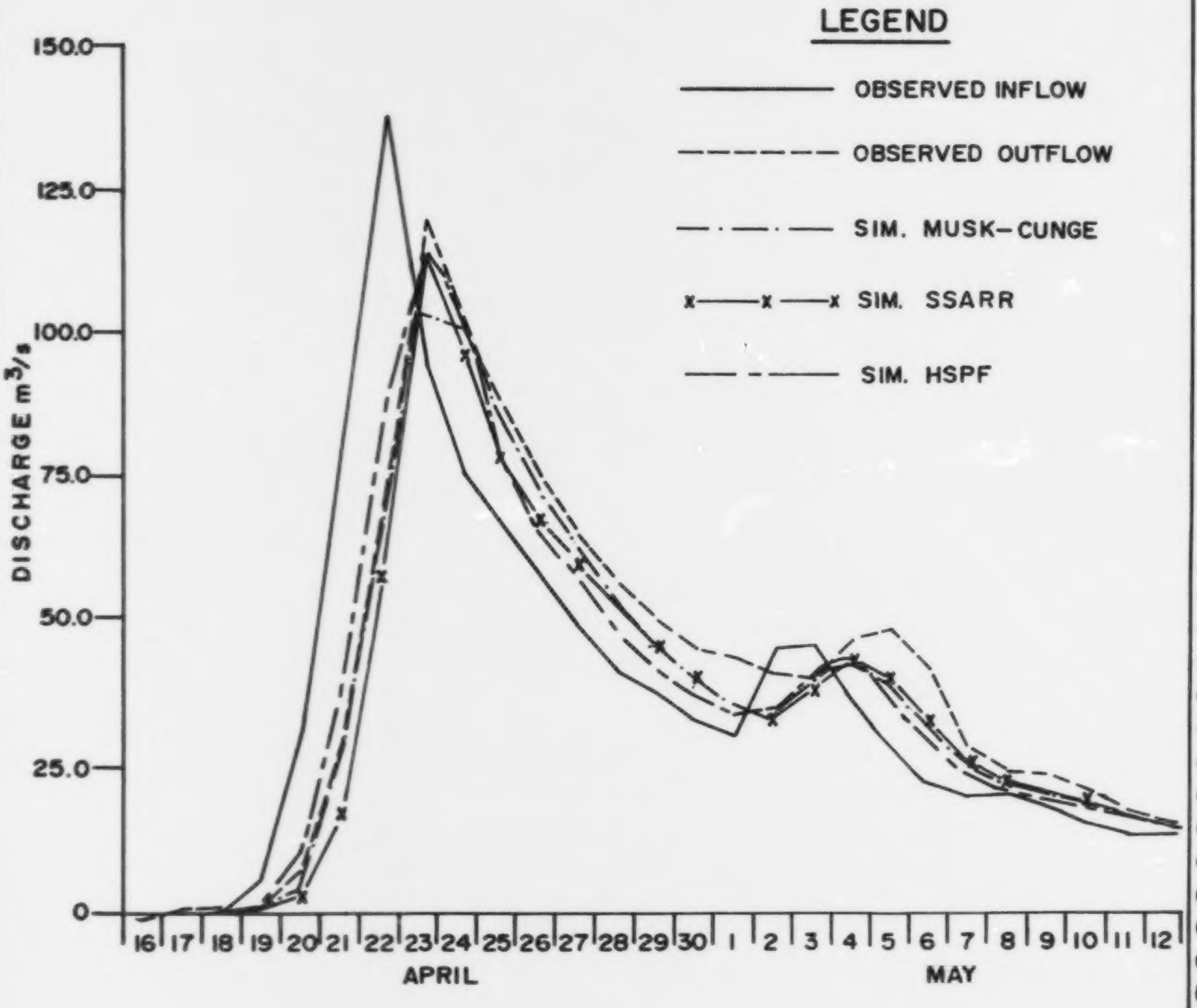

FIGURE 28

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

### TRIAL ROUTINGS

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Natural Resources
Water Resources

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CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
AGREEMENT RESPECTING FLOOD FORECASTING
PHASE TWO
APPLICATION AND EVALUATION

# APPENDIX A

# ADAPTING the HSPF MODEL to the PRAIRIE ENVIRONMENT

G. E. Mohr, P. Eng.

Environment Canada Manitoba Natural Resources

Winnipeg, Manitoba
February, 1988

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# Chapter 1

# INTRODUCTION

In March, 1981 the governments of Canada and Manitoba entered into a five-year "Agreement Respecting Flood Forecasting" to examine river flow forecasting methodologies for Manitoba. A major component of the study was the testing of the HSPF hydrologic model (Johanson et al.,1984), first on the Boyne River Basin, then on three other tributary watersheds in southwestern Manitoba.

This appendix is a compendium of topics dealing with the adapting of the HSPF model to conditions found in southern Manitoba. It does not cover general procedures used in applying HSPF as this can be found in the main report and in the literature (Donigian and Davis, 1978 and Donigian et al.,1984). Rather, the attempted solutions to specific problems encountered in applying HSPF to the prairies are discussed. As such, it may be useful to other HSPF users who are applying HSPF to similar prairie watersheds.

The appendix has two main sections: summer modelling and spring runoff modelling. This division matches the calibration process followed in this study whereby the model was first calibrated for the summer period, and then calibrated for the following spring period.

# Chapter 2

# SUMMER MODELLING

## 2.1 Introduction

The primary aim of the modelling effort under the Agreement Respecting Flood Forecasting was to simulate spring snowmelt runoff. Simulation accuracy during the summer period was of lesser importance. However the summer calibration was required in order to calibrate the model parameters which control the movement of water in and out of the soil storages. These parameters were then used in the spring simulations. Furthermore, the simulated fall soil moistures from the summer calibrations were used as starting conditions for spring runoff simulations.

Calibration is a two step process requiring the initial estimation of the parameters that define the runoff process for both the land and the stream channels, followed by adjustment of these parameters. The literature (Donigian and Davis, 1978 and Donigian et al.,1984) documents the normal procedures used to calibrate the HSPF model. However the separate calibration of the summer and spring periods which was undertaken in the present study presented problems in estimating certain starting parameters. Difficulties were encountered in estimating channel properties due to the lack of cross section data. The attempted solutions to these problems are discussed in the following sections.

In addition, it was found during the calibration of the process parameters that certain processes were not being modelled correctly. The parameter adjustments that were applied to compensate for these problems are discussed in section 2.3, "Fine Tuning".

## 2.2 Estimating Watershed Parameters

In calibrating the lands portion of the HSPF model the values for 28 parameters had to be determined. The majority of the parameters were set to HSPF's initialized default values or the values suggested by other modellers. Default values were used for processes that were relatively constant from basin to basin in the study, or where the model was found to be rather insensitive to a change in the parameter, or where it was difficult to isolate the effects of a change in a given parameter. The half dozen variables that remained to be estimated can be divided into two general categories; physical parameters which are measurable and, process parameters that can only be determined through calibration.

The channel routing portion of the HSPF model requires the determination of four parameters that define the geometric and hydraulic parameters of the channel.

### 2.2.1 Starting Values for Upper and Lower Zone Storage

The recommended model calibration procedure is to simulate over a continuous period of a few years, establishing a water balance on an annual basis (Donigian et al.,1984). This approach eliminates the need to manually compute starting storages every season. Furthermore the starting values may be roughly estimated since errors in these values would be dampened out after a couple of months and become inconsequential when the model is run continuously for a number of years. In this study, however, calibration was carried out on discrete summer periods from May through October for selected years. This procedure was chosen so as to reduce the volume of data which had to be prepared. The selected approach does require a more accurate estimate of start up values for May 1, because errors would not be damped out quickly enough, especially in years when a large portion of the summer runoff occurred in May and June.

Guidelines were developed for estimating startup values for upper zone storage (UZS) and lower zone storage (LZS). The start up value for upper zone storage (UZS) was chosen by categorizing the spring runoff as either low, medium or high and correspondingly setting the starting value at 50%, 75% and 100% of the nominal storage. Special circumstances were also considered when initializing the UZS. For example, the occurrence of a rainfall event of an inch or more on the recession limb of the spring hydrograph or the summer simulation starting less than 15 days after the spring snowmelt peak would raise the value of UZS into the next higher bracket.

The start up value for lower zone storage (LZS) was the sum of water held in the lower zone at freeze-up the previous fall and the volume of water added in the springtime. This was computed using the soil moisture measured in the previous fall at a soil moisture monitoring site to calculate the volume of water held in the top 430 mm. of soil and adding a portion of the observed precipitation in the spring. Rain falling on the rising limb of the spring hydrograph or soon thereafter would not be added to the lower zone due to the low meltwater absorption capability of frozen soil. Rain occurring after the peak would fall on an upper zone saturated by meltwater and would cause the transfer of water to the lower zone. As such, if a significant precipitation event occurred during this period, the LZS computed for the fall would be increased by an appropriate amount.

### 2.2.2 Estimating Channel Routing Parameters

HSPF's channel routing requires information on channel cross section, bed slope and an estimate of channel roughness. For the Phase One study, surveyed cross sections were available. However for the streams studied under Phase Two there was little survey data available, which meant that a means of estimating the required data had to be derived.

Huang (1978), describes a technique by which theoretical cross-sections of simple geometric shape can be developed using the information available on topographic maps. In essence, it is assumed that the channel slope as measured on the map (contour interval/channel length between intervals) would be an adequate approximation of the energy slope and that channel geometry is simple enough to be described by either a rectangle or a triangle. For the rectangular channel assumption the channel would have vertical banks distance 'D' apart and the water depth should be much less than the channel width. For the triangular channel assumption the distance 'D' would be the channel width at a point where the channel depth is equal to the contour interval. Also uniform side slopes are assumed.

An investigation of the available survey data, as well as the pertinent literature, indicated that a triangular shape would be the preferred approximation for small tributary streams in Manitoba.

The determination of 'D' based solely upon the information available for the study basin turned out to be a significant problem. Huang assumed that it was possible to determine the required information simply through the use of topographic maps. For that to be possible the contour lines must exhibit the classic pattern which identifies a river valley with the contour lines bending or looping in the upstream direction. Also the available maps must have sufficient detail to allow the required measurements.

Of the three basins examined in this study, only Gopher Creek and its tributaries flow within valleys of sufficient depth that the width 'D' could be distinguished on the available topographic maps (1:50,000). Elgin and Willow Creeks are located on till plains of much gentler topography and do not have well developed stream valleys. Although the downstream ends of these streams are somewhat incised, the depth of the stream valleys are less than the contour intervals. As a result, the routing parameters for both Willow and Elgin Creeks had to be estimated from information derived from the few surveyed cross-sections that were available.

## 2.3 Fine Tuning

This section discusses other changes made to the modelling process during the summer modelling, each of which represented a small improvement in the overall simulation performance. The topics discussed under this heading are; variable monthly lower zone evaporation parameter (LZETP), variable upper zone nominal storage parameter (UZSN) and channel priming.

### 2.3.1 Variable Monthly Lower Zone Evaporation Parameter

The lower zone evaporation parameter, LZETP is used to compute actual evapotranspiration from the lower soil moisture zone. The initial estimate of LZETP was based on the fraction of the watershed with deep rooted vegetation. Suggested values for this parameter (which is defined as K3 in the ARM's Model, the precursor of the HSPF model (Donigian et al.,1978)) range from 0.25 for open land and grassland to 0.7 - 0.9 for heavy forest. Monthly values of LZETP are employed to account for variations in transpiration due to plant growth.

A table of monthly LZETP was prepared using a consumptive water use table from Garg (1976). The values were computed assuming that the transpiring crop would be wheat, the start of planting would be May 15, and the growing season would be 90 days. For each year simulated, the LZETP values for the planting and harvesting months were then adjusted from their initial estimates based on the average planting and harvest dates for that year for southwestern Manitoba as compiled by Manitoba Department of Agriculture. The LZETP parameter was changed on a specific date using the "SPEC-ACTION" feature of the model which allows variables to be changed during execution.

### 2.3.2 Variable Upper Zone Nominal Storage Parameter

It was observed during the Phase One study (Canada-Manitoba Flood Damage Reduction Program (1985)) and preliminary work in Phase Two that the HSPF model would over-simulate runoff for rainstorms that occurred in September and October. Donigian et al.(1977) report poor simulation results following tillage operations. In south western Manitoba tillage is generally carried out after harvesting and prior to freeze-up. Tillage results in increased capacity in the near surface soil zone. To account for tillage the value of UZSN was increased following harvesting using the SPEC-ACTION feature of the model. This change resulted in improved simulations for the fall.

### 2.3.3 Channel Priming

All three streams studied under Phase Two are ephemeral and stop flowing in the summer after a dry spell of two to three weeks. When flows recommence the initial runoff is trapped in local pools in the stream bed before the water reaches the downstream end of the channel. This process is referred to as channel priming.

To account for channel priming, a dead storage volume was added for each channel. However after adding dead storage it was found that the storage did not empty as quickly as was hoped. The only method for emptying dead storage is through evaporation. To "speed up" the emptying of the dead storage the evaporation rate was arbitrarily doubled. It was rationalized that the evaporation rate could be doubled because of the large transpiration that occurres along the channel from trees and shrubs which are not explicitly accounted for in the model.

# Chapter 3

# SPRING RUNOFF MODELLING

## 3.1 Introduction

The accuracy with which the snowmelt runoff process can be modelled is affected by many factors. The spatial representativeness of data is a major concern during hydrologic modelling and has been discussed to some degree in the main report under the individual basin writeups and in the general discussion. In addition, Donigian et al.(1984) identified deficiencies in modelling algorithms (infiltration and snowmelt) as a major problem in the modelling of snowmelt runoff using HSPF.

Frozen ground affects meltwater infiltration and is not explicitly accounted for in HSPF's infiltration algorithms. Section 3.2 discusses the methods employed in modelling frozen ground.

The snowmelt algorithms that were originally developed for the HSPF model have gone through a period of refinements by a number of Canadian researchers (Male and Granger, 1979; Price, 1977; MacLaren Plansearch, 1984). Section 3.3 discusses these refinements and their application in the current study.

Solar radiation data was not available in close proximity to any of the basins modelled in this study. It therefore had to be synthesized based on solar radiation data recorded at Winnipeg. An empirical relationship was developed which adjusted the solar radiation at Winnipeg in accordance with the difference between bright sunshine recorded near the basin and at Winnipeg. The procedure used is described in Appendix D of the Phase One Report.

During the current study this procedure was found to overestimate computed basin radiation when Winnipeg was sunny and the basin was under a dull overcast (Mohr,1988). This may have contributed additional error to snowmelt runoff estimates in some situations. However this problem was discovered too late to permit modification of the procedure.

## 3.2 Meltwater Infiltration

The infiltration process during snowmelt is affected by frozen soil which restricts meltwater absorption by the soil. Granger et al.(1984) have observed in Saskatchewan that snow meltwater was confined to an average depth of 26 cm. (with a standard deviation of 10 cm.). They contend that the shallow depth can be attributed to refreezing of meltwater in the frozen soil. From this work Gray et al.(1985) classified the snowmelt infiltration process into three categories, namely; Unlimited, Limited and Restricted.

- Unlimited - a soil containing a high percentage of large, air-filled, non-capillary pores or macropores at the time of melt and most or all the snow water infiltrates. Examples of these soils are dry, heavily-cracked clays and
- Limited - infiltration is governed primarily by the snow cover water equivalent and the frozen water (ice) content of the 0-300 millimetres soil layer.
- Restricted - infiltration is impeded by an impermeable layer, such as an ice lense on the soil surface or at a shallow depth in the soil. For practical purposes the amount of meltwater infiltration can be assumed to be negligible and most of the snow water goes to evaporation or direct runoff.

Initial spring simulation runs showed that spring runoff was underestimated for all of the basins for all years simulated. Clearly, for the years simulated the unlimited scenario being employed in the model did not correctly represent actual processes in the watersheds. To further test the frozen ground theory, the HSPF model was set up to duplicate the limited and restricted cases as illustrated in Figure 1.

The limited case was modelled as having only overland flow and interflow and required two parameter changes. Infiltration was reduced to a minimum value of 0.0001 mm. The interflow/overland flow relationship is a function of the infiltration parameter and it was, therefore, necessary to readjust the relationship to approximately the same value as the unlimited case. This required changing the interflow (INTFW) parameter from 2.5 to 400.0 mm.

The restricted case was modelled as having only overland flow and this required interflow to be reduced to the model's minimum value. This was accomplished by changing the interflow (INTFW) parameter to 0.0001 mm. while keeping infiltration (INFILT) at 0.0001 mm.

Testing of the limited and restricted infiltration scenarios showed that the model performed best for the limited case.

Calibration of the depressional storage was important for the proper simulation of spring events under the limited condition. A series of runs were made with the start up value for upper zone storage (UZS) kept at zero and the nominal value of upper zone storage (UZSN) varied between 25 and 62 mm. This gave a sufficient range in depressional storage to adequately simulate meltwater absorption by the soil. Simulated runoff volumes were compared with the observed runoff volumes to identify the value which produced the best result for each of the years under consideration. These were then averaged to produce the best overall estimate of UZSN in terms of spring runoff volumes.

A different methodology was used on Gopher Creek. On Gopher Creek it was decided to test whether the ICE FLAG option in the HSPF model in combination with a lower infiltration rate (33% of its summer value) could be used to approximate the affect of ground frost. The ICE FLAG option is used to simulate the refreezing of water which would otherwise leave the pack. If ice forms at the bottom of the pack HSPF would reduce the infiltration equation by a prorated amount. With 25 mm. of ice at the bottom of the pack infiltration would be reduced by 90 percent. The infiltration rate would increase once the ice started to melt.

Both methods performed reasonably well for the basins studied, simulating within plus or minus 15 percent of the observed runoff volume over the spring period. Donigian et al.(1984) suggests that based on their experience a difference of less than 10% between simulated and recorded should be judged as very good. Donigian's assessment was largely based on monthly and annual volumes and was meant for non-snowmelt events. Donigian suggests that the general accuracy of snowmelt runoff calibration is not as rigorous as for non-snowmelt events because the former suffers more from non representative meteorological data and model deficiencies. Also Water Survey of Canada attests to larger errors in estimating streamflow under ice breakup conditions than under open water conditions. The frozen ground modifications can therefore be considered to have produced good results.

## 3.3 Snowmelt Algorithm Modifications

Most of todays snowmelt algorithms can be traced back to the work of the US Army Corps of Engineers (1956). From this work Anderson and Crawford (1964) and Anderson (1968) developed one of the first snowmelt routines as an adjunct of the Stanford Watershed Model IV (Crawford and Linsley, 1966) the grandfather of the HSPF model.

MacLaren Plansearch (1984) made substantive changes to HSPF's snowmelt algorithms using data from a well instrumented research basin in southern Ontario. The changes made to HSPF by Maclaren were:

1. to allow for solar radiation absorption by ground cover,
2. to provide for a faster albedo decay for a shallow snowpack,
3. to simplify the model's negative heat storage calculation,
4. to add a new convection and condensation equation based on airmass stability above the snowpack using the Richardson number,

5. to remove the change to the albedo formula that occurs on April 1, and
6. to modify the longwave melt equation.

The first four items were radical changes to the snowmelt code. All the changes, except for No. 6, were incorporated into the Manitoba Water Resources Branch (MWRB) version of the model.

Further changes made by MWRB included :

1. the addition of a detailed printout of the snowmelt computations,
2. the use of the complete longwave formula with cloud cover incorporated into the formula,
3. the modification of the negative heat storage calculation using a variable maximum heat loss,
4. the modification of the albedo decay formula to maintain relatively high values prior to snowmelt and to produce a faster decay during active melt, and
5. the restriction of melt when the air temperature is below zero degrees celcius.

The original version of the HSPF model (Johanson et al.,1984) controlled snowmelt by varying only two factors; the solar shading of the snowpack (SHADE) which accounts for vegetation cover, and a field adjustment factor to the convection and condensation equation (CCFACT). Field research done by MacLaren Plansearch (1984) has resulted in additional snowmelt variables being added. These consist of a modified solar algorithm and a new convection and condensation equation based on the Richardson turbulence number. Testing of the convection equation showed that at times the Richardson convection equation produced melt rates three times as high as the original convection equation. The difference was discussed with Price(1985) the researcher that worked with MacLauren in modifying the snowmelt equation. He couldn't explain why there would be such a large difference. Since the equation couldn't be verified with actual prairie data it was decided to use the original snowmelt equation.

MacLaren also made changes to the snowmelt algorithm based on measurement of the snowpack's reflectance (albedo) and observations of the amount of snowmelt caused by solar radiation. The original albedo decay equation was a time decay function developed from observations in mountainous areas. This in essence produced a constant rate of albedo decay with no consideration of either the snowpack depth or temperature. Many researchers have found that this relationship results in an albedo that is too low prior to melt and that decays too slowly during active melt for shallow snowpacks. MacLaren introduced a change to increase the decay rate during active melt when the snowpack decreased to a specified depth. However, in the current study in which simulations were started four to six weeks prior to melt a means was required to keep the albedo high prior to melt. The solution was to reduce the decay rate prior to melt and increase the decay rate when the snowpack temperature was at or above zero.

Many researcher's have noted that for shallow snowpacks of less than 20 centimeters solar radiation is able to penetrate to the underlying ground surface where it is absorbed. MacLaren observed that when the snowpack was less than 20 cm. the HSPF model would overestimate the solar radiation gain of the snowpack. Maclaren found that the portion of radiation absorbed by the ground could be modelled using an exponential equation with snow depth as the independent variable. This was incorporated into the version of the model used in the study. The MWRB version now models radiation absorbed by the snowpack as a function of snow depth and two "dullness" factors based on pack temperature.

In the original melt code, convective energy transfer from the snowpack to the atmosphere is not modelled, because normally insufficient energy exists at the surface to sustain the process due to the poor conductivity of snow. However, conditions can develop when radiation imparts sufficient energy to the surface for the convective energy transfer to be outgoing from the pack. Instead of modelling this outgoing energy, radiation melt was restricted from occurring until the air temperature was above freezing.

Detailed documentation of the FORTRAN code changes to the HSPF snowmelt code are contained in an internal MWRB report (Mohr,1988 (2)).

# Bibliography

[1] Anderson, E.A., (1968), "Development and Testing of Snow Pack Energy Balance Equations", *Water Resources Research*, 4(1):19-37.

[2] Anderson, E.A. and Crawford, N.H., (1964), *The Synthesis of Continuous Snowmelt Runoff Hydrographs on a Digital Computer*, Tech. Rep. No. 36., Dept. of Civil Eng., Stanford Univ., Stanford, California, 103 p.

[3] Canada-Manitoba Flood Damage Reduction Program, (1985), *Agreement Respecting Flood Forecasting; Phase One, Planning and Design* (unpublished).

[4] Crawford, N.H. and Linsley, R.K.,(1966), *Digital Simulation in Hydrology: Stanford Watershed Model IV*, Stanford Univ., Dept. Civil Eng. Rep. 39.

[5] Donigian, A.S., Beyerlein, D.C., Davis, H.H., and Crawford, N.H., (1977) *Agricultural Runoff Management (ARM) Model Version II: Refinement and Testing*, EPA-600 Runoff Management (ARM) Model, EPA-600/3-77-098, Environmental Research Laboratory, Office of Research and Development, USEPA, Athens, Georgia.

[6] Donigian, A.S. and Davis, H.H., (1978) *User's Manual for Agricultural Runoff Management (ARM) Model*, EPA-600/3-78-080, Environmental Research Laboratory, Office of Research and Development, USEPA, Athens, Georgia.

[7] Donigian, A.S., Imhoff, J.C., Bichnell, B.R. and Kittle, J.L.(1984), *Application Guide for Hydrological Simulation Program - FORTRAN (HSPF)*, EPA-600/3-84-065, Environmental Research Laboratory, Office of Research and Development, USEPA, Athens, Georgia.

[8] Garg, S.K., (1976), *Irrigation Engineering and Hydraulic Structures*, Khana Publishers, New Delhi, India.

[9] Granger, R., Gray, D.M. and Dyck, G.E. (1984), *An Infiltration Model for Frozen Soils*, Paper 84-313 presented to 1984 Annual Meeting, Canadian Society of Agricultural Engineers, August 12-23, Winnipeg, Man.

[10] Gray, D.M., Landine, P.G. and McKay, G.A., (1985), *Forecasting Streamflow Runoff from Snowmelt in a Prairie Environment*, Proceedings from the Canadian Society of Civil Engineering, Saskatoon, Saskatchewan, pp. 213-231.

[11] Huang, Y.H., (1978), "Channel Routing by Finite Difference Method", *Journal of the Hydraulics Division*, Vol. 104, No. HY10, Proc. Paper 14088, pp. 1429-1445.

[12] Johanson, R.C., Imhoff, J.C., Kittle, J.L. and Donigian, A.S., (1984) *Hydrological Simulation Program - Fortran (HSPF): User's Manual for Release 8.0*, EPA-600/3-84-066, Environmental Research Laboratory, Office of Research and Development, USEPA, Athens, Georgia.

[13] MacLaren Plansearch, (1984), *Snow Hydrology Study Phases I and II: Study Methodology and Single Event Simulation.* Report submitted to Conservation authorities and Water Management Branch, Ministry of Natural Resources, Province of Ontario, Toronto, Ontario.

[14] Male, D.H. and Granger, R., (1979), "Energy and Mass Fluxes at the Snow Surface in a Prairie Environment", *Procedure of Modelling of Snow Cover Runoff*, S.C. Colbeck and M. Ray, Editors, U.S. Army Corps of Engineers., Cold Regions Research and Engineering, New Hampshire, pp. 101-124

[15] Mohr, G.E., (1988), *Accuracy of Synthesized Basin Solar Radiation*, Unpublished internal report, Manitoba Water Resources Branch, Winnipeg, Manitoba.

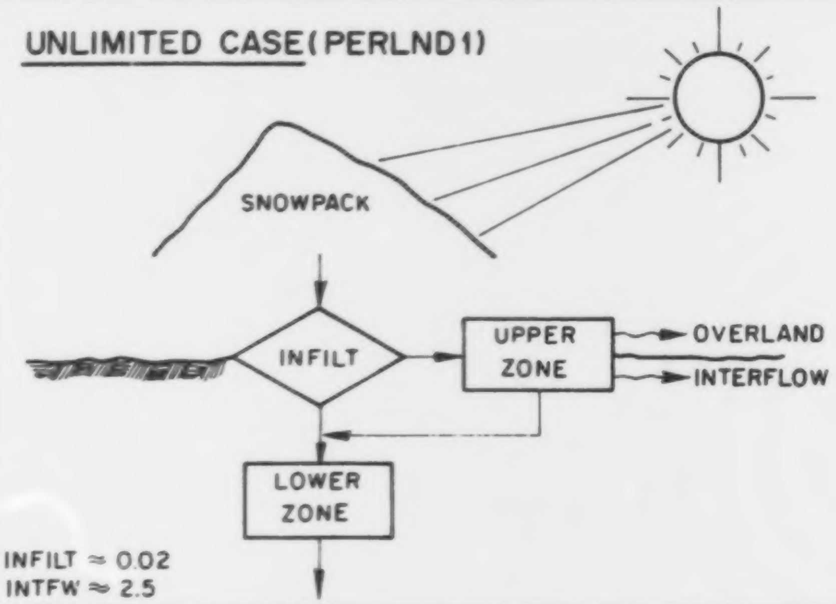
[16] Mohr, G.E., (1988)(2), *Modifications to HSPF's FORTRAN Code for Snowmelt*, Unpublished internal report, Manitoba Water Resources Branch, Winnipeg, Manitoba.

[17] Price, A.G., (1977), *Snowmelt Runoff Processes in a Sub-Artic Area*, Sub-Artic Research Paper No. 29, Climatological Research Series No. 10, Department of Geography, McGill University.

[18] Price, A.G., (1985), pers. comm.

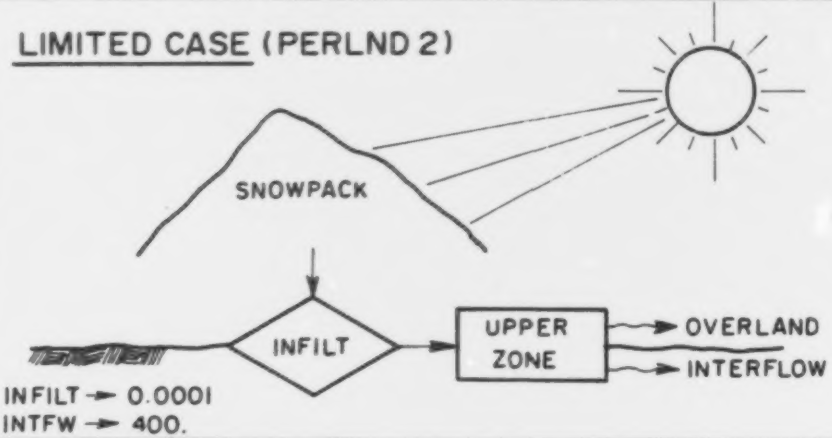
[19] U.S. Army Corps of Engineers, (1956) *Snow Hydrology, Summary Report of the Snow Investigations*, U.S. Army Corps Engineers, North Pacific Div., Portland, Oregon.

## UNLIMITED CASE (PERLND 1)


SNOWPACK
INFILT
UPPER ZONE
OVERLAND
INTERFLOW
LOWER ZONE
INFILT ≈ 0.02
INTFW ≈ 2.5

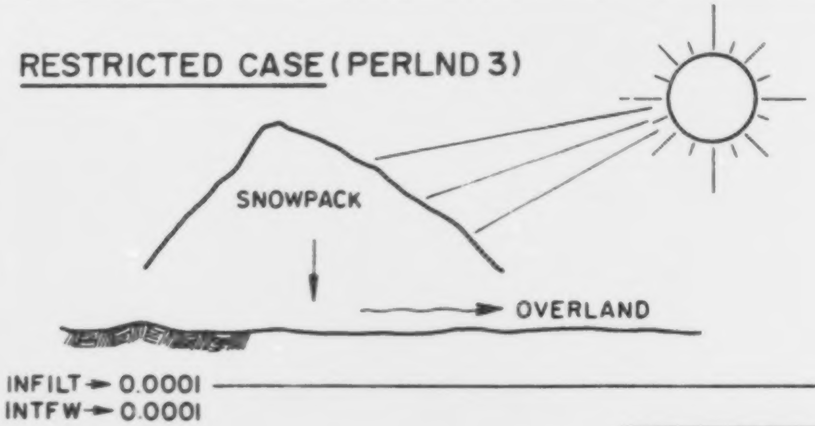
- STUBBLE FIELDS WITH A DRY FALL AND NO WINTER MELT.
- HSPF INDICATOR, LOW UPPER (UZS) + LOWER (LZS) ZONE STORAGE IN THE FALL.

## LIMITED CASE (PERLND 2)


SNOWPACK
INFILT
UPPER ZONE
OVERLAND
INTERFLOW
INFILT → 0.0001
INTFW → 400.

- STUBBLE FIELDS (WET FALL OR MID-WINTER MELT)
- SUMMER FALLOW (DRY FALL)
- HSPF INDICATOR, AVERAGE VALUES OF UZS AND LZS.

## RESTRICTED CASE (PERLND 3)


SNOWPACK
OVERLAND
INFILT → 0.0001
INTFW → 0.0001

- SUMMER FALLOW (WET FALL OR MID-WINTER MELT)
- HSPF INDICATOR, VALUES OF BOTH UZS AND LZS NEAR NOMINAL.

FIGURE 1

CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM

FLOOD FORECASTING AGREEMENT-PHASE TWO

# FROST MODIFICATION OF HSPF INFILTRATION PROCESS

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CANADA-MANITOBA FLOOD DAMAGE REDUCTION PROGRAM
AGREEMENT RESPECTING FLOOD FORECASTING
PHASE TWO
APPLICATION AND EVALUATION

# APPENDIX B

# ROUTING STUDIES

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# Chapter 1

# LITERATURE REVIEW

## 1.1 Introduction

A flood wave is a mass of water moving through a stream channel, superimposed on the normal mass of moving water. This can best be visualized as a very long and low wave causing a momentary change in the local depth of water. As such, water is displaced above and below the mean water level, which results in a weight or gravity force being exerted that will cause the wave to move and deform, irrespective of the movement or lack of movement of the fluid below it.

Flood routing is the tracing by calculation of the movement of a flood wave between two points in a river channel. The passage of the flood wave at any point in the channel can be represented by a hydrograph of discharge or stage versus time. Flood routing models attempt to reproduce the characteristic changes which will occur to the upstream hydrograph as a result of the passage of the flood wave to the downstream location. The downstream hydrograph will be displaced in time by an amount equal to the time it takes for the flood wave to travel between the discrete locations and the peak flow will be reduced or attenuated by the resistance and storage characteristics of the channel.

A successful routing is one where the downstream hydrograph can be accurately predicted when only the upstream hydrograph is known, or in other words, one that is able to simulate the change in shape and displacement in time of the flood wave hydrograph as it passes between the upstream and downstream points.

## 1.2 The Idealized Flood Wave

Two types of wave movement have been conceptualized and mathematically proven. Both are present to some degree in a flood wave.

A kinematic wave or disturbance is one where the discharge and velocity characteristics are a function of the depth of flow alone and where these relationships do not change over time. This implies that the slope of the water surface and the bed slope are approximately equal and that the changes in water surface slope which will normally produce a looped rating curve are insignificant during the passage of a kinematic wave. As such, the flow characteristics of a kinematic wave can be adequately described by uniform flow concepts.

Kinematic waves can only propagate in the downstream direction with a speed that approximates to:

$$V_w = \frac{1}{B}\frac{\partial Q}{\partial y} \tag{1.1}$$

where: $V_w$ is the speed of the flood wave relative to the banks,
$B$ is the surface width,
$Q$ is the discharge,
and $y$ is the flow depth.

Since the kinematic wave speed is a function of depth alone, flow velocity will increase with increaseing depth of flow. This will mean that the flood peak will be travelling at a higher velocity than the smaller magnitude flow which precedes it. Any kinematic wave must, therefore, advance with the face becoming continually steeper, as shown in the Figure 1.1.

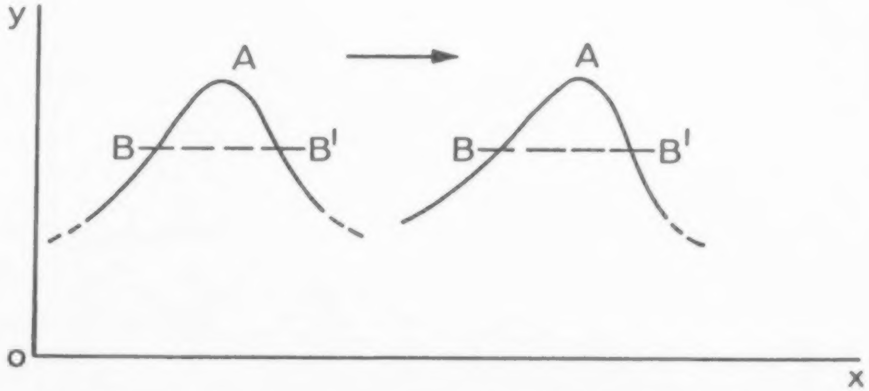

**Figure 1.1: Distortion of the Kinematic Wave Profile with Downstream Movement**

Any horizontal chord, such as $B - B'$ does not lengthen or disperse and as a result the wave cannot subside and the crest (A) maintains the same maximum depth. This result accords well with the common observation that flood waves in steep rivers of uniform section do not subside as they move downstream.

Dynamic waves are those having the characteristics of small gravity waves. A gravity wave propagates in response to a gravity force acting upon a mass of fluid which extends above the mean water level. Wave fronts will propagate away from the point of highest depth, in both the upstream and downstream directions. The speed at which these will propagate is equal to $\sqrt{gy}$, where $g$ is the gravity force and $y$ is the depth of flow. In a stream channel the wave fronts are superimposed on a mass of moving fluid and will, therefore, appear to be moving at speeds of $V^{+}_{-}\sqrt{gy}$ relative to an observer on the bank, $V$ being the mean velocity of the streamflow.

Dynamic wave motion is the result of many more forces than kinematic waves. As a result, a more complete description of the flow which includes these forces is required. The equation of motion used to describe dynamic waves is based on Newton's second law (force = mass $x$ acceleration) and the principles of conservation of energy and momentum. The additional terms, which are described as secondary slope terms and the flow conditions under which each is required are shown in the following example of the equation of motion for one dimensional flow (Henderson, 1966).

$$S_f = S_o - \frac{\partial x}{\partial y} - \frac{\partial v}{\partial g}\frac{\partial v}{\partial y} - \frac{1}{g}\frac{\partial v}{\partial t} \qquad (1.2)$$

steady uniform flow
steady non-uniform flow
unsteady non-uniform flow

where: $S_f$ is the friction slope,
$S_o$ is the channel bed slope,
$x$ is horizontal distance,
$y$ is vertical stream depth,
$v$ is flow velocity,
$t$ is time,
and $g$ is gravitational acceleration.

The additional terms can be described as:

The energy slope for unsteady flow = $S_e$

$$S_e = \frac{\partial H}{\partial x} \qquad (1.3)$$

$$\text{or} \quad S_e = \frac{\partial z}{\partial x} + \frac{\partial x}{\partial y} + \frac{V}{g}\frac{\partial v}{\partial x} \qquad (1.4)$$

and the acceleration slope = $S_a$

$$S_a = \frac{1}{g}\frac{\partial v}{\partial t} \qquad (1.5)$$

which describes the energy loss or gain due to the unsteadiness of the flow.

where: $H$ is the total energy head,
$z$ is the height of the stream bottom above datum,
and the other terms are as previously defined.

By multiplying each of the terms which make up the energy slope by the gravity term we can identify the component forces.

$g\partial z/\partial x$ is the gravitational force accelerating the fluid in the direction of flow and $g\partial y/\partial x$ and $v\partial v/\partial x$ describe the changes in kinetic and potential energy.

The additional slope terms induce dispersion, which causes the downstream dynamic disturbance to attenuate as it travels downstream.

In any natural flood wave both kinematic and dynamic wave movement may be present. As the bed slope term is usually by far the most important term, even if the other slope terms are not negligible, the main bulk of the fluid moves substantially as a kinematic wave. In particular the speed of the flood wave may be expected to approximate that of a kinematic wave. Unless the other slope terms are negligible however, the character of the flow will be modified by the dynamic disturbance. In particular, the development of the wavefront profile is controlled by the relative strengths and positions of the two disturbances.

The major characteristics of a kinematic wave are the same as the monoclinal wave illustrated in Figure 1.2. At some distance downstream the flow is yet unaffected by the flood wave. It has a depth $y_2$, velocity $v_2$, a discharge $Q_2$ and an area of flow $A_2$. Upstream of the wave front the depth of flow has increased to $y_1$, the velocity to $v_1$, discharge to $Q$ and the area of flow to $A_1$.

During time interval $t$, the wave front moves forward a distance equal to $V_w t$. When the wavefront passes over the flow in the channel it takes in a steady discharge $Q_o = (V_w - V_1)A_1$ at the front. Since the wave configuration in this example is of stable form and constant volume, the wave front leaves behind an equal steady discharge $Q_o = (V_w - V_2)A_2$ to the upstream side. The steady discharge which flows through the wavefront in the upstream direction is termed the overrun.

The velocity of the wavefront is directly related to the change in discharge which results from the passage of the flood wave by $V_w = (Q_1 - Q_2)/(A_1 - A_2)$. If there is flow in the channel before the passage of the flood wave $V_W$ will be greater than either $V_1$ or $V_2$ . In the case of a flood wave entering a dry channel $V_w = V_1$. The critical depth of flow which would result from a steady discharge equal to the overrun is the overrun critical depth $y_c$ .

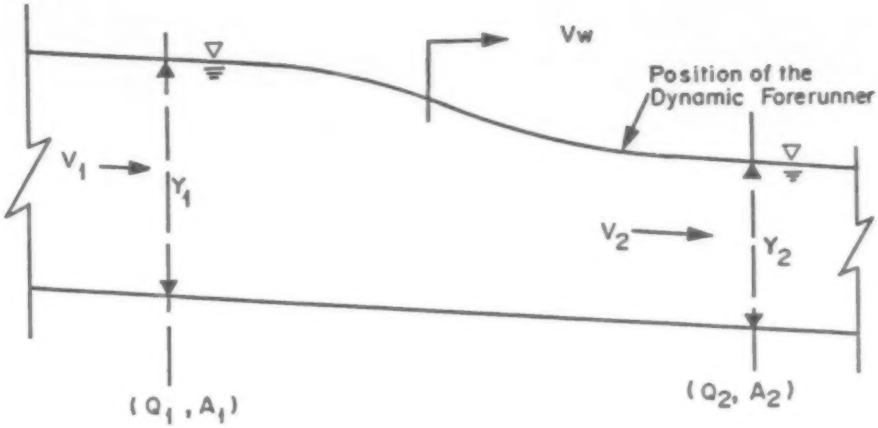

**Figure 1.2: The Monoclinal Flood Wave**

The dynamic forerunner will propagate at a speed of $V_2 + \sqrt{gy}$. This will be greater than the speed of the kinematic disturbance except in the case of mountain torrents. As such, it will be found at some distance down the leading edge of the flood wave shortly after its formation and this distance will increase over time until theoretically it is an infinite distance downstream. Unless the rate at which flow depth increases exceeds a foot every few minutes however, a positive dynamic disturbance of this type will attenuate rapidly as it moves downstream.

Since the dynamic forerunner will generally attenuate to a scarcely perceptible feather edge a short distance downstream it would seem to make little contribution to the progress of the flood wave as a whole. It does however, play an important role in helping to control the development of the wave front profile.

It has been stated that the main bulk of the flood wave approximates a kinematic wave. If it were exactly kinematic the friction slope would equal the bed slope and from the usual form of the resistance equations, wave speed would increase with depth and discharge. Any true kinematic disturbance must advance with the face becoming continually steeper, as illustrated in Figure 1.1. This would ultimately cause the formation of a surge or vertical wave front if not for the fact that as the wave becomes steeper the secondary slope terms found in the dynamic equation of motion become increasingly important. The effect of these terms is to introduce dispersion and attenuation not found in the kinematic wave proper. This retards the steepening of the wave front, thus delaying or preventing all together the formation of a surge.

Initially the wave front is neither stable nor constant, kinematic wave motion acts to steepen the wave front while the dynamic disturbance causes dispersion and lengthening of the wave. It is only when the two types of wave motion are in balance that a stable profile can be developed. In a natural channel this will not occur until the wave has travelled a considerable distance, in the order of hundreds of kilometers. The stable wave front profiles which can develop are a monoclinal wave and a surge. It is possible to state the conditions under which the formation of either might occur in terms of either the relative speeds of the kinematic and dynamic disturbances or the hydraulic conditions responsible for these velocities.

If the velocity of the dynamic forerunner is greater than that of the kinematic disturbance the steepening process associated with the kinematic disturbance will be arrested short of surge formation. As shown in Figure 1.3(a), when $V_2 + \sqrt{gy} > V_w$, $y_c$ must be less than y and the profile will be theoretically of infinite length with the depth of flow asymptotic to the sections of uniform flow upstream ($y_1$) and downstream ($y_2$). The stable form which would result must be a smooth steady-state profile described as a monoclinal wave. Lighthill and Whitham proposed the alternative name "kinematic shock" for this wave front since it is ultimately derived from the steepening of a kinematic wave.

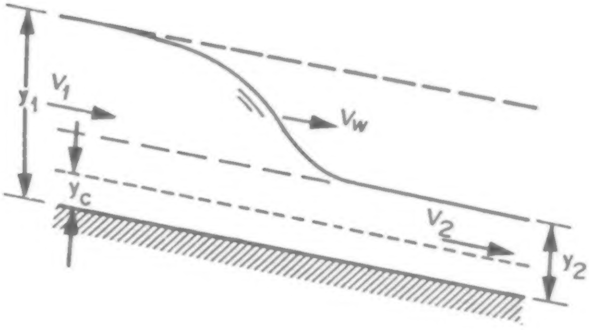

Figure 1.3(a): Profile of Monoclinal Flood Wave
when $y_c < y_2$ or $V_w < V_2 + \sqrt{gy_1}$

On the other hand, if the velocity of the kinematic disturbance is greater than that of the dynamic forerunner ($V_w > V_2 + \sqrt{gy}$) the wave front will continue to steepen until the kinematic wave front and the dynamic forerunner occur together. For this to occur the overrun critical depth ($y_c$) must be greater than $y_2$ and a surge will form with the wave front profile vertical at the overrun critical depth line. This is illustrated in Figure 1.3(b).

If there is flow in the channel prior to the passage of the flood wave, this condition could not be met unless the rate of water depth rise was about one foot every few minutes. This would only occur under a catastrophic natural condition or through the operation of man-made controls. These criteria also explain why any release of flow into a dry channel must take the form of a surge since $y_2 = 0$.

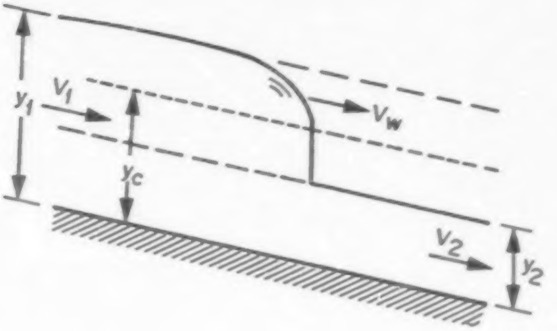

**Figure 1.3(b): Profile of Monoclinal Flood Wave when $y_c > y_2$ or $V_w > V_2 + \sqrt{gy_1}$**

## 1.3 Attenuation of a Flood Wave

During the passage of a flood wave, the peak discharge and maximum stage measured at an upstream location will be greater than that measured at a downstream station.

The reduction in peak stage and discharge which results from the flood wave travelling down the stream channel is referred to as attenuation or subsidence.

Attenuation is controlled by the resistance and acceleration terms in the dynamic equation of motion and a much simpler mechanism, the pondage or storage of water in the irregularities of a natural river channel, having the effect of a chain of small lakes.

Attenuation of a flood wave is an example of a general principle which can conveniently be expressed in electrical terms: a pulse fed into a system can be attenuated either by a capacitance or by a resistance.

The strength of the dispersion introduced by the dynamic disturbance is a function of the relative importance of the secondary slope terms in relation to the bed slope term. Typical values for each of the slope terms taken from an actual river in steep alluvial country are:

| | $S_o$, | $\partial y/\partial x$, | $v/g(\partial v/\partial x)$, | $1/g(\partial v/\partial t)$, |
|---|---|---|---|---|
| *ft/mile* | 26, | 0.5, | $0.125 - 0.25$, | 0.05 |

These figures relate to a very fast rising flood where the flow increased from 283 $m^3/sec$ (10,000 $ft^3/sec$ ) to 4247 $m^3/sec$ (150,000 $ft^3/sec$ ) and decreased to 283 $m^3/sec$ again, within a 24 hour period (Henderson, 1966). Even in this case, where the acceleration terms are comparatively large, the last three slope terms are so small that only bed slope need be retained i.e. on steep slopes only $S_o$ is significant. Subsidence as a result of dynamic effects will be relatively small.

Dynamic effects become increasingly important if either the bed slope term is small or if the flow is highly supercritical.

When the bed slope is small, the secondary slope term $\partial y/\partial x$ can easily be of the same order of magnitude as the bed slope ($S_o$), although the ratio $S_o/S_w$, where $S_w$ is the slope of the wave surface, is invariably greater than unity and usually greater than 10. In a uniform channel the amount of dispersion attributable to the term $\partial y/\partial x$ is small. The Froude number in this case will be very low so that the third slope term will be negligible. Furthermore it can be shown that the third and fourth terms are of the same order of magnitude and are of little significance. Therefore, in the case of a stream with a mild bed slope $S_f = S_o - \partial y/\partial x$

In the case of highly supercritical flow the dynamic wave front steepens due to the general steepening of the profile characteristic of kinematic wave motion. The dynamic wave celerity increasingly diverges from the kinematic wave celerity with a resultant increase in the importance of the energy and momentum terms. These flow conditions only exist in mountain torrents and are of little interest on the prairies.

Henderson (1966) concludes that "if the wave is 'mild' (i.e. slow rising, as natural floods are) then substantial rates of subsidence and departures from the kinematic wave velocity can only be produced by storage effects, such as in the level pool routing".

The storage of a portion of the flood wave's volume is an important mechanism contributing to the attenuation of the flood peak and in many cases is the primary mechanism by which attenuation occurs. The most common storage concept is the storage in a lake or reservoir, where the rate of inflow is more or less unrestricted but the rate of outflow is subject to some form of control. There will be a change in the volume of water in storage at any time when the rates of inflow and outflow are not equal ie. water will be entering storage at any time the rate of inflow exceeds the rate of outflow.

This change in storage volume must equal the difference in the volume of inflow and the volume of outflow over the time period in question in order to satisfy the principle of conservation of mass. This principle is commonly expressed as the equation of continuity in the form:

$$\Delta S = (Q_I - Q_O)t \tag{1.6}$$

where: $S$ is the storage volume,
$Q_I$ is the inflow discharge,
$Q_O$ is the outflow discharge,
and $t$ the time period in seconds.

A lake or reservoir has hydraulic properties which make the computation of storage volumes relatively simple. The water surface is horizontal and thus both the rate of outflow and the storage volume are functions of the water depth and are directly related. These functions are considered to be constant and are unaffected by differences in the rates of inflow and outflow.

The concept of storage is also important in a natural stream channel where irregularities in the channel and flood plain can act as a series of small lakes. Determining the storage within a stream channel is more complex since the water surface is not horizontal. The water surface in this case has a characteristic slope which is variable during the passage of a flood event. Storage within the reach is no longer uniquely related to outflow or the flow depth at the downstream end of the reach, but is the product of both the upstream and downstream depths, or the rates of inflow and outflow. As this is the case, it is usual to subdivide storage within a stream channel into prism storage, dependant upon outflow alone, and wedge storage dependant upon the difference between inflow and outflow. This concept acknowledges the fact that during the passage of a flood the water surface slope is no longer constant or parallel to the bed slope as would be the case under steady uniform conditions. The volume of water in the reach above a line which is parallel to the bed at the outflow depth, is considered to be in wedge storage. The water below this line is in prism storage.

## 1.4 Classification of Flood Routing Models

The classification of routing models can be difficult due to the assumptions implicit in the various models and variations in the computation methods.

The simplest type of routing method is purely empirical and is based on intuition and observations of past flood waves. Relationships between upstream and downstream stages or discharges are developed over many observations. This type of routing does not explicitly describe either the storage or resistance characteristics of the stream and due to the difficulty in deriving the empirical relationships are of limited use.

Storage or hydrologic routing techniques assume that storage effects are the predominant mechanism causing attenuation of the flood wave. As such, they are based on the concept of conservation of mass and do not directly include the effects of resistance to the flow. This allows the routing to be achieved using simple computational methods.

The most basic form of a storage routing technique is a level pool routing where it is assumed that the water surface is horizontal and that depth, storage and outflow are all uniquely related. This is commonly used for routing through a lake or reservoir. The main element in the calculation is the balancing of inflow, outflow and the volume of water in the lake using the continuity equation. A secondary equation relates the outflow conditions to the depth of storage.

Storage routing methods are also used for routing a flood wave down a stream channel. In this case the problem of defining the relationship between the volume of storage and flow conditions is more complicated. As has been discussed previously, the storage volume within a river channel is not uniquely related to outflow or the water depth at the downstream end of the reach. It is a product of both the downstream depth, which is used to define the volume of prism storage, and the difference between the upstream and downstream depths, used to define the volume held as wedge storage. Both types of storage must be accounted for in some manner when a storage routing method is applied to the channel routing problem.

A concept similar to the level pool routing can be applied to routing through a river channel by conceptualizing the river channel as a series of connected reservoirs. By keeping the length of each reservoir sufficiently short the modeller is able to reduce the amount of water held as wedge storage to an insignificant volume. Routing through the series of reservoirs which represent the river channel can then be achieved using the simpler concepts of a level pool routing.

In this application storage volume for a given downstream flow depth is computed as the volume of water below a line which parallels the stream bed (prism storage) instead of the volume below a horizontal elevation, as would be the case for a reservoir.

A unique relationship is then assumed between the depth at the downstream end of the reach and both the storage volume and the outflow rate. Since this implies uniform flow conditions at the outlet it also allows the application of uniform flow formulae, such as Manning's (1891), to define the outflow conditions where necessary.

Another approach used in hydrologic or storage routing methods is to develop a function which accounts for the effect of wedge storage. One of the more common hydrologic routing techniques which uses this approach is the Muskingum method. This method addresses the problem of defining the storage volume by assuming a linear relationship between storage and flow in the form:

$$V = K\left[O + X\left(I - O\right)\right] \tag{1.7}$$

where: $V$ is the storage volume,
$O$ is the outflow volume,
$I$ is the inflow volume,
$K$ is the storage parameter term,
and $X$ relates the relative importance of the inflow and the outflow.

(Henderson, 1966)

In this case the volume of water held as prism storage is defined as K (O) and the volume of wedge storage as $KX(I - O)$.

Hydrologic or storage routing models tend to be applied more commonly than other routing methods due to the relative simplicity of the routing computations and relatively small data requirements.

At the other extreme of the scale in terms of computational complexity is the broad category of routing models generally called "wave models" These utilize some form of the equation of motion to describe the channel's resistance characteristics and the movement of the flood wave and a continuity equation to account for changes in storage. Although much more complicated than simple hydrologic techniques these models overcome the inherent inability of storage routing methods to simulate changes in the water surface profile due to the passage of a flood wave. Depending on the form of the equation of motion employed they can also provide information on the characteristics of the flow, such as velocities and flow depths at many locations along the channel and at anytime during the simulation.

Wave models tend to be subdivided based on the assumptions made by the model. These assumptions are attempts to identify the dominant processes of wave movement and attenuation in order to simplify the routing computations. The assumptions made by the model can only be determined by identifying which terms in the equation of motion are included and the computation method employed.

The full equation of motion for one dimensional flow (Equation 1.2) can be conveniently rewritten as:

$$\overset{\text{I}}{\frac{1}{g}\frac{\partial v}{\partial t}} + \overset{\text{II}}{\frac{v}{g}\frac{\partial v}{\partial x}} + \overset{\text{III}}{\frac{\partial y}{\partial x}} + \overset{\text{IV}}{(S_f - S_o)} \quad = \quad 0 \tag{1.8}$$

The terms can be named as follows: (I) is the local inertia term, (II) is the convective inertia term, (III) is the pressure differential term and (IV) is simply the bed and friction slopes.

The dynamic wave model utilizes the full equation of motion. The wave models which utilize approximations of dynamic waves and the terms included in each are:

| | |
|---|---|
| Dynamic Wave | I + II + III + IV |
| Gravity Wave | I + II + III |
| Steady Dynamic Wave | II + III + IV |
| Diffusion Wave | III + IV |
| Kinematic Wave | IV |

(Miller, 1984)

Another category of routing models are those which attempt to account for some of the dynamics of flood wave movement while utilizing much simpler computational methods than is possible with a full wave model. To achieve this goal these methods use analogies or further approximations of wave characteristics to produce results which are similar to those of the wave models. The advantage in such an approach lies in either a simplified computation method or easier determination of the necessary flow parameters.

An example of such an approach is the Muskingum-Cunge method. This method is an adaptation of the basic Muskingum routing model to produce results comparable to those of the diffusion-wave model. The basic Muskingum model is able to attenuate the flood peak due to a numerical dispersion inherent in the finite difference method of the routing computations. This has been called an arbitrary diffusion since the amount of diffusion is not directly related to any physical parameters. Cunge identified the magnitude of this diffusion with that of the corresponding term in the diffusion-wave model. In this way he developed a methodology for computing the parameters of the Muskingum method from the physical characteristics of the channel and the flow, using the parameters of the diffusion wave model. This adaptation allows the routing computations to be as simple as those of the Muskingum model while considering some aspects of the dynamic nature of the flow.

Although it might seem desirable to be able to put models into specific categories, this can be both misleading and cause improper evaluation of a model's potential. A case in point may serve to illustrate this.

The **basic Muskingum** model is a **hydrologic model** as it concentrates on the concept of storage and does not evaluate the physical parameters controlling the movement of water. Cunge (1969) evaluated the amount of numerical dispersion introduced by the Muskingum model in terms of the dispersion of the convective-diffusion model, thus **Muskingum-Cunge** can be called a **second order approximation of the convective diffusion model**. As the method of determining the routing parameters utilizes the actual channel and flow characteristics of the reach it can also be appropriately called a **hydraulic model**.

If the routing parameter $X$ is set at its upper limit (0.5) and wave celerity evaluated from a steady flow rating function, Muskingum-Cunge reduces to a **kinematic wave** model and will result in pure translation of the wave (Weinmann and Laurenson, 1979). At the other extreme if X is set at 0.0 the model will behave as a level routing, a pure **hydrologic model**. Ponce and Yevjevich, 1978, present a version of this model which uses variable routing parameters. This allows Muskingum-Cunge to account for the non-linearity of the flow phenomenon, which, according to Weinmann and Laurenson would elevate this model to the status of a **kinematic model corrected for dynamic effects**.

It can be seen that the classification of models, except in very general terms, requires familiarity with the computational method, the procedure used to calculate the routing parameters and any modification to the model or its application by the individual modeller.

## 1.5 Selection of a Routing Model

The selection of a channel routing model for a particular application is influenced by the relative importance one places on the following factors: (1) model accuracy; (2) the accuracy required in the given application; (3) the type and availability of required data; (4) the available computational facilities; (5) the computational costs; (6) the extent of flood wave information desired; (7) one's familiarity with a given model; (8) the extent of documentation, range of applicability, and availability of a "canned"or packaged routing model; (9) the complexity of the mathematical formulation if the routing model is to be totally developed from "scratch" (coded for computer); and (10) one's capability and time available to develop a particular type of routing model. Taking all factors into consideration, and recognizing that each application may change the relative importance of each factor, it is apparent that there is no universally superior routing model. (D. L. Fread in Hydrological Forecasting, 1985)

Flood wave movement has been shown to be the result of both dynamic and kinematic wave movement while attenuation is also affected to a large degree by storage effects. The selected model must account for any mechanism which has a significant effect on either the timing or attenuation of the flood wave. Whether or not a mechanism will be considered as significant will depend on the desired accuracy of the routing and the nature of the flood wave. In many cases one or more terms of the equation of motion can be excluded if the loss in accuracy is outweighed by a reduction in computational complexity.

The routing model selected should depend on the required output parameters and the quality of simulation required by a particular study. Hydrologic models are only able to generate information on discharge or water levels at end of reach locations. If flow velocities or water level profiles are required a more sophisticated model will have to be used. Similarly, if the generated data is to be applied where very accurate information is critical, as in the case of flood proofing a large urban centre, it might be advisable to use the most sophisticated model possible. The model selected should produce the necessary data outputs with the minimum of data input and computation time.

The quantity and quality of available input data can be a serious restriction to the application of a routing model. The most sophisticated model will not produce quality results if the input data are inadequate. As a general rule, as the models become more sophisticated their data requirements increase accordingly. Some hydrologic models can be calibrated and applied using only daily flow data while models which employ some form of wave motion equation require fairly large amounts of quality data on both the stream channel and flow characteristics in order to be calibrated. The model selected will have to be compatible with the available data.

All routing models attempt to simulate or describe only a portion of the mechanisms responsible for the movement and attenuation of the flood wave. Some concept of storage must be included in each model in order to satisfy the principle of conservation of mass. The major difference between models is in the method by which the movement of the flow is accounted for. Hydrologic models use a simplified analogy of the equation of motion in the form of a storage-outflow curve, some form of uniform flow equation, or simply a mathematical relationship such as that used in the Muskingum form of routing. This approach cannot simulate the changes in the stage-discharge relationship which result from the passage of a flood wave. Hydrologic models, therefore, will only produce an accurate simulation in cases where attenuation is caused primarily by storage effects and where outflow characteristics do not diverge significantly from uniform flow conditions.

By identifying the slope terms included in the routing equations of models based on approximations of dynamic waves it is possible to determine the flow conditions each model is capable of simulating. Various researchers have developed criteria to aid in the selection of the most appropriate model. Henderson (1963) presented a qualitative criterion which is useful for understanding the general concept of how an appropriate routing model should be selected. Bed slope was differentiated into steep, intermediate and mild. Movement of flood waves on steep slopes can be adequately described by kinematic wave models. On steep slopes, those capable of producing supercritical flow, the flood wave speed ($V_W$) exceeds the speed of the dynamic disturbance if the Froude number of the flow is greater than two. Under these conditions the secondary slope terms do not affect wave movement and can, therefore, be eliminated from the routing equation. The friction slope is assumed to be essentially equal to the bed slope.

On mild slopes a positive dynamic disturbance attenuates rapidly so the effect of slope terms describing inertial effects will be negligible. The flood wave can be adequately described by the diffusion-wave model which includes the pressure-differential term ($\partial y/\partial x$) in addition to the bed and friction slopes.

Flood waves in channels of intermediate slope may be altered by all of the secondary slope terms. Models which use the full equation of motion such as the dynamic wave model are appropriate for describing the flood wave under these conditions.

Many other authors have suggested criteria based on a quantitative analysis of specific flow characteristics (Miller and Cunge, 1975; Lighthill and Whitham, 1955; Ponce, Li and Simons, 1978; etc.). The criteria developed tend to be applicable only in very specific cases and, therefore, not useful for decision making.

Many other model characteristics can affect the accuracy of the simulation. If the desired accuracy cannot be achieved, it may be necessary for the modeller to examine the internal workings of the model in order to understand the assumptions inherent in the model and the possible sources of error.

# Chapter 2

# SIMPLIFIED ROUTING METHODS

## 2.1 Introduction

During the course of the study several routing methods have been examined for their utility to route floods on tributary streams. Methods which concentrate on storage concepts as the primary mechanism causing attenuation (hydrologic methods) were found to be more suitable than those utilizing the full dynamic equations.

Dynamic models are capable of producing more accurate and detailed results than the simplified models if sufficient high quality streamflow and stream channel data is available. However, they have dimensional requirements that necessitate a very dense computational network to simulate flows on smaller streams. The time required to calibrate the model and the computational costs were not justified when the data limitations and resultant accuracy were considered.

The basic Muskingum method was not included in the routing trials due to difficulty of determining the routing parameters except by a trial and error calibration process. The Muskingum-Cunge routing method uses the same routing equation but provides a method of computing the routing parameters from stream channel and flow characteristics. This eliminates much of the uncertainty associated with the calibration of the standard method. As such, Muskingum-Cunge is commonly acknowledged to be an improvement over the standard method and was used for the routing trials.

The Muskingum method is still the standard by which most simple routing methods are compared due to its widespread use. As such, a detailed description of the method is presented before those of the three simplified routing methods (Muskingum-Cunge, SSARR, and HSPF) used in the tributary routing trials. The results of these trials and a brief description of the models are given in the main report.

## 2.2 Muskingum Flood Routing

### 2.2.1 Introduction

The Muskingum method of flood routing was proposed by McCarthy (1938) and is one of the best known and widely used hydrologic methods. It is a hydrologic model as it concentrates on the concept of storage and does not directly evaluate the physical parameters controlling the movement of water. This method addresses the problem of defining the storage volume by assuming a linear relationship between storage and flow.

Due to the simple mathematics involved, this method can be applied using hand calculations and a trial and error calibration method. As such, it was especially suitable for routing floods down poorly documented rivers before the advent of high speed computers. Although the basic Muskingum method continues to be widely, used most researchers advocate that it be replaced by more accurate routing models which use stream channel characteristics to determine the routing parameters.

### 2.2.2 Development of the Method

The Muskingum routing method is based on a continuity equation which expresses the principle of conservation of mass, in the form:

$$\frac{dS}{dt} = (I_t - O_t) \tag{2.1}$$

where: $S$ is storage, $t$ is time, $I$ is inflow, and $O$ is outflow.
The dynamic equation of motion has been replaced by a simple linear relationship in the form:

$$S_t = K\left[X I_t + (1 - X) O_t\right] \tag{2.2}$$

where: $K$ and $X$ are the Muskingum routing parameters.
This expression is commonly called the Muskingum storage equation. The parameter $K$ is called the storage time constant and is approximately equal to the time of travel through the reach. The relative importance of inflow and outflow in determining the storage within the reach is defined by $X$.

The Muskingum routing equation is derived by combining the continuity equation in the finite difference form:

$$\frac{S_2 - S_1}{\Delta t} = 0.5(I_1 - I_2) - 0.5(O_1 - O_2) \tag{2.3}$$

With the Muskingum storage equation, to obtain the routing equation:

$$O_2 = C_1 I_1 + C_2 I_2 + C_3 O_1 \tag{2.4}$$

The routing coefficients are defined as:

$$C_1 = \frac{KX + 0.5(\Delta t)}{K - KX + 0.5\Delta t} \tag{2.5}$$

$$C_2 = \frac{-KX + 0.5(\Delta t)}{K - KX + 0.5\Delta t} \tag{2.6}$$

$$C_3 = \frac{K(1 - X) - 0.5(\Delta t)}{K - KX + 0.5\Delta t} \tag{2.7}$$

For the routing equation to be physically valid certain dimensional criteria must be met. The first of these is that the routing coefficients must be positive as well as adding up to one, in order to ensure mass conservation. In addition the routing interval $(\Delta t)$ has been loosely constrained by Chow (1964, Handbook of Applied Hydrology) as satisfying the condition:

$$2KX \leq \Delta t \leq K \tag{2.8}$$

and by Viessman et al (1977, Introduction to Hydrology) as:

$$K/3 \leq \Delta t \leq K \tag{2.9}$$

As a result of these conditions there are finite limits on the range of routing parameters that can be used, especially if $(\Delta t)$ is a fixed value. For example, during the spring flood period there is generally only one discharge value per day available for most of the small streams in Manitoba. As such $K$ is limited $\geq 1$ day in order to meet the criteria outlined by either Chow or Viessman et al. Routing flows on streams where the time of travel, between the end of reach locations, is less than one day would require that either more frequent discharge values be obtained or a certain degree of inaccuracy accepted.

Assuming that $\Delta t = 1$ day and $K \geq 1$ day there are only a limited number of values for $X$ which will satisfy the criterion of the routing coefficients being positive values, those where $KX \leq 0.5\Delta t$ or in this case $KX \leq 0.5$.

$$\begin{array}{lll} if: & K = 1.0 & 0 < X < 0.5 \\ & K = 2.0 & 0 < X < 0.25 \\ & K = 3.0 & 0 < X < 0.167 \\ & K = 4.0 & 0 < X < 0.125 \\ & K = 5.0 & 0 < X < 0.100 \end{array}$$

The effect of various values of $X$ can be more clearly illustrated using the some simple examples with $X$ set at its maximum and minimum values of 0.5 and 0.0.

If $X = 0.5$, the Muskingum storage equation would become:

$$S = K[0.5(I) + 0.5(O)] \tag{2.10}$$

and both inflow and outflow contribute equally to the storage volume. This results in pure translation of the wave (i.e. kinematic wave behavior) corresponding to fully distributed storage, as in a reach whose physical properties are uniform throughout its length. Values of $X < 0.5$ introduces a numerical distortion, similar to a diffusive term, which produces an attenuation of the flood wave.

If $K$ and $\Delta t$ are both equal to 1, and $X = 0.5$, the routing expression becomes:

$$O_2 = 1.0(I_1) + 0.0(I_2) + 0.0(O_1) \tag{2.11}$$

The net effect of this expression is that there is no attenuation but the hydrograph will be lagged one routing period.

If $X = 0$, the Muskingum storage equation would become:

$$S = K[0.0(I) + 1.0(O)] \tag{2.12}$$

and storage is directly a function of outflow alone. The value of $X = 0.0$ corresponds to a fully concentrated or reservoir type action where inflow causes an instantaneous response, the principle effect being the attenuation of the inflow peak. The effect of $X$ can be further illustrated by the use of the routing expression.

If $K$ and $\Delta t$ are both equal to 1.0, an $X$ of 0.0 will produce the expression:

$$O_2 = 0.333(I_1) + 0.333(I_2) + 0.333(O_1) \tag{2.13}$$

and inflow will produce an immediate response at the downstream boundary as in a reservoir. In addition the peak flow will be attenuated.

### 2.2.3 Application to a Specific Channel Reach

Calibration of the Muskingum routing model usually involves the making of initial parameter estimates and then a trial and error process to determine which routing coefficients produce the best simulation. Several methods of computing the routing parameters can be found in the literature. In order to illustrate some of the limitations to the routing method and the methods of parameter estimation, 1979 spring flows on the Boyne River between Stephenfield and Carman were routed using the parameter estimates produced by some of these methods.

The first method (Ponce and Yevjevich) assumes that wave celerity (c) can be approximated by:

$$c = \frac{\partial Q}{\partial A} \text{ and as such } \mathrm{K} = \frac{\Delta \mathrm{X}}{\mathrm{c}} \tag{2.14}$$

The $X$ parameter is computed as:

$$X = 0.5\left[1 - \left(\frac{q}{S_o c \Delta x}\right)\right] \tag{2.15}$$

where:
$Q$ is discharge
$A$ is flow area
$q$ is unit width discharge
$x$ is reach length
$S_o$ is bed slope

For this example values of discharge, flow area and top width were obtained from metering notes for the rising limb of the hydrograph at the respective gauging stations. Wave celerity and unit width discharge were computed for each station and then averaged in order to obtain values representative for the whole reach. This resulted in values of c = 4.27 $\text{ft}^3/\text{sec}$ and $q = 39.96\ \text{ft}^3/\text{sec}/\text{ft}$ . at the discharge peak.

Using these numbers $K = 42{,}661$ seconds or 0.49 days and $X = 0.454$ and the routing coefficients were computed as:

$$\begin{aligned} C_1 &= 0.941 \\ C_2 &= 0.362 \\ C_3 &= -0.303 \end{aligned}$$

Although the $K$ parameter and the routing coefficients violate the conditions for physical validity a routing was performed as a check of the calibration technique. The use of these coefficients results in both lag and attenuation being underestimated, with discharges on the rising limb of the hydrograph approximately double the recorded values and attenuation of the peak 33 % of observed. The use of a negative $C_3$ coefficient also causes distortion of the pattern of flows.

In order to satisfy the limiting conditions stated, $K$ was increased to one day (86,400 sec.), C becomes equal to 2.11 ft/sec and $X = 0.408$. This produced routing coefficients of:

$$\begin{aligned} C_1 &= 0.832 \\ C_2 &= 0.084 \\ C_3 &= 0.084 \end{aligned}$$

This improved the simulation somewhat by reducing the hydrograph distortion as well as improving the timing of the hydrograph although attenuation of the peak flow is still underestimated.

A second calibration technique suggests the simultaneous determination of $K$ and $X$ by solving the expression:

$$K = \frac{0.5\Delta t\left[(I_1 + I_2) - (O_1 + O_2)\right]}{X(I_1 - I_2) + (1 - X)(O_1 - O_2)} \tag{2.16}$$

This expression is derived by combining the Muskingum storage and continuity equations in a finite difference form.

The routing period was maintained as one day for this example. Starting with $K$ equal to one day it was not possible to solve the expression for most of the rising limb of the inflow hydrograph.

As a final test of the routing method a trial and error calibration procedure was attempted. The value of $K$ was increased to two days and $X$ was found to range from 0.0 for the first simulation period, to 0.50 by the routing period after the outflow peak. As it has already been stated for a $K$ of two days and a routing period of one day, $X$ cannot exceed 0.25 if the routing equation is to be physically valid. As such 0.25 was considered the upper limit of $X$.

Routings performed using an $X$ of 0.0 and 0.25 both proved unsatisfactory. With $X$ set at 0.0 the simulated hydrograph tracked extremely well except near the outflow peak. Attenuation of the peak discharge was nearly three times that observed. Increasing $X$ to its maximum reduced peak attenuation slightly but also caused the simulated flows to be excessively lagged.

These simple exercises demonstrate the basic weakness of the Muskingum method of routing for tributary flows; the use of one set of routing parameters throughout the modelling period. This is not a problem if the model is used as originally intended, to predict the lag and attenuation of only the peak discharge.

These results cannot be considered conclusive, since local inflows and suspected problems with recorded discharges tend to obscure the differences between observed and simulated outflows. Notwithstanding, it tends to indicate that the routing parameters which will accurately simulate the initial rise of the 1979 Boyne River spring hydrograph will cause the peak discharge to be underestimated. Similarly, if the routing parameters are adjusted to simulate the peak discharge, the routed hydrograph will not track properly before the peak. This would create difficulties if the model were used operationally for flood forecasting since there would be no way to confirm the accuracy of the simulation during the early stages of the hydrograph's rise.

The main reason for this inaccuracy is the use of a single value for the $K$ parameter, which is an expression of wave speed. The observed results confirm the fact that the mean velocity of the flow at low discharges is less than the velocity at high flows. Since $K$ is constant and dependant upon a reference discharge (usually the average of the peak discharges upstream and downstream) it cannot possibly be representative of the velocities at all stages during the passage of a flood wave, especially in smaller tributary streams where there is generally a fast rising hydrograph. This problem exists for any modification of the Muskingum method, except for those which recalculate the routing parameters for each routing period (e.g. Ponce and Yevjevich (1978), Miller and Cunge (1975), etc.).

Other problems seem to exist in the calibration techniques. The use of flow characteristics to determine wave celerity $c = \partial Q/\partial A$ is of questionable value if celerity changes dramatically over the range of flows contained in the hydrograph. In addition, accurate metering notes must be available over a range of flows in order to apply this method of calibration.

The conditions under which Muskingum should be applied is also restricted by the need to satisfy the physical validity requirements. For example, if the time of travel between locations is less than the modelling period the Muskingum should not be applied since the dimensional limits of the model have been violated. This will be obvious since a negative coefficient will result.

## 2.3 Muskingum-Cunge Flood Routing

### 2.3.1 Introduction

The Muskingum-Cunge flood routing model uses the same routing equation as the original Muskingum model. It differs, however, in the method used to compute the routing parameters.

The basic Muskingum model induces attenuation as a result of its finite difference scheme. This has been termed a mathematical or even an arbitrary diffusion since it is not based on a directly measurable physical parameter.

Cunge identified the magnitude of this diffusion with that of the corresponding term in convective-diffusion equation. In this way, Cunge was able to define the parameters of the Muskingum method in terms of the parameters for the diffusion method. This included the development of a methodology for logically relating the physical characteristics of the channel and the flow to these terms.

This method of computing the routing parameters allows the attenuation of the peak discharge to be estimated with a high degree of precision while maintaining the computational simplicity of the original Muskingum model. It has the additional advantage that it does not have the difficulties in dealing with tributary inflows which the diffusion methods have.

### 2.3.2 Development of the Model

The theory behind the improvements to the Muskingum model can be found in Cunge (1969) or in the Flood Studies Report (1975) which contains a brief description and Fortran program for applying the method. This report concentrates on the application of the method and presents only the briefest description of the theory behind it.

Cunge observed that the Muskingum routing equation can be written in a finite difference form form, using K defined as equalling the reach length divided by average speed of the flood peak ($\omega$). The derived expression is a finite difference representation of the kinematic wave equation.

Once $K$ was defined it remained to calculate $X$. Cunge observed that by expressing $(I^n)$ in terms of their Taylor expansions, it could be shown that $X$ could be defined in terms comparable to the diffusion method.

$$\begin{aligned} \text{when } \mu &= (0.5 - X)\,\omega\Delta x && (2.17) \\ \text{as } \mu &= d\frac{\overline{Q_p}}{L} && (2.18) \\ \text{it follows that X} &= 0.5 - \left(\frac{d\overline{Q}_p}{L\omega\Delta x}\right) && (2.19) \end{aligned}$$

where: $\mu$ is the diffusion parameter, $L$ is the length of the whole reach divided into subreaches of length $(\Delta x)$, $d$ is the attenuation parameter corresponding to $(\overline{Q_P})$ the mean peak discharge through the reach, and is the speed of the flood wave.

The attenuation parameter is computed using the mean channel width or flooded area, channel length, and channel slope. Once $K$ and $X$ are determined, the routing coefficients are computed using the standard relationships for the Muskingum method.

In order to apply this method over a range of flow magnitudes graphical relationships are developed between peak discharge and the parameters used to compute the routing coefficients so that the modeller can quickly select appropriate values.

### 2.3.3 Application to a Specific Channel Reach

The application to a particular river reach is also covered by the Flood Studies Report. In essence the method involves using the physical characteristics of the stream channel to define the attenuation parameter for at least the largest recorded flood and a bankfull flood. The characteristics of observed hydrographs are used to define time required for these flood events to travel through the reach $(T_p)$. The speed of the flood waves is defined as $(\Delta x/T_P)$. The curvature of the upstream discharge hydrograph at the peak is used is used as an indicator of the peakiness of the event and to compute a correction factor for the kinematic wave speed.

Once this procedure has been applied to the largest recorded flood, the inbank flood, and any other recorded floods, the attenuation of the hydrograph $(Q^*)$ is computed for each event. The method is constrained by the requirement that $Q^*/Q_p < 0.1$, where $Q_p$ this condition is not met a new $Q^*$ is computed.

The values of wave speed and the attenuation parameter corresponding to the values of mean peak discharge are plotted and curves are drawn through the computed points. This is difficult if there are very few observed floods and requires some judgement as to the inflection points. The values of mean peak discharge can be estimated using the computed attenuation as $Q_p = Q_p - 0.5Q^*$ or by averaging the observed upstream and downstream peak discharges for the events, if they are known to be without significant lateral inflows. To route a flood the values of wave speed and the attenuation parameter corresponding to the mean peak discharge $\overline{Q_p}$ are read off the graph and input into the computer program. The computed attenuation $Q^*$ is used to estimate the mean peak discharge through the reach when only the upstream peak is known. The program computes values of $K$ and $X$ and the routing coefficients before proceeding with the routing.

## 2.4 SSARR Routing Method

### 2.4.1 Introduction

The channel routing method used in the SSARR model was developed for modelling the Columbia River basin (Rockwood, 1958), and was called a "multiple-phase reservoir storage channel routing" by the author. The routing method was based on the concept that water input to the basin moves through a series of storage reservoirs which delay basin runoff, starting with the storage on the ground surface, and continuing through lake, reservoir, and channel storages. Each of these acts to delay or lag the input hydrograph. If the time delay inherent in each of these storage reservoirs could be defined, it would be possible to simulate the lag and attenuation caused by the storages. The channel routing method is applicable to routing where storage is a function of outflow, such as in lakes or short channel reaches, where wedge storage is negligible in comparison with prismatic storage.

### 2.4.2 Development the Routing Equation

The SSARR routing method utilizes a continuity equation expressed in the form:

$$I_t = O_t + \frac{dS}{dt} \tag{2.20}$$

where: $I$ and $O$ are inflow and outflow rates at time $t$
and $dS/dt$ is the rate of change in storage at time $t$.

For cases where storage is a function of outflow:

$$S = T_s(O) \tag{2.21}$$

where $T_s$ is the proportionality factor between storage and outflow.
Differentiating Eg. 2.15 with respect to time:

$$\frac{dS}{dt} = T_s\left(\frac{dO}{dt}\right) \tag{2.22}$$

Substituting this into equation 2.20, it becomes:

$$I_t = O_t + T_s\left(\frac{dO}{dt}\right) \tag{2.23}$$

$$\frac{dO}{dt} = \frac{(I_t - O_t)}{T_s} \tag{2.24}$$

which is the form of the storage equation used in the original method.

In the SSARR model the routing is performed for each time increment t. The equation must therefore be put into a usable form for computing outflow at the end of the modelling period.

By manipulating the basic storage routing equation and letting,

$$T_s = \frac{(S_2 - S_1)}{(O_2 - O_1)} \tag{2.25}$$

the working equation, for computing the outflow at the end of the period ($O_2$) is:

$$O_2 = \left(\frac{I_m - O_1}{T_s + t/2}\right) t + O_1 \tag{2.26}$$

where: $I$ = mean inflow $=[(I_1 + I_2)/2]$,
$O_1$ = outflow at the beginning of the period,
$t$ = time duration of computation period,
and $T_s$ = time of storage.

In the computer program the value of $t/(T_s + t/2)$, is evaluated for a specific condition, and multiplied by the difference between $I_m$ and $O_1$ to obtain the change in outflows. The outflow ($O_2$) computed from the first incremental routing is then saved and converted to initial outflow ($O_1$) for the next routing period.

### 2.4.3 Determination of Ts

It can be seen that $T_s$ is the principal routing parameter. For natural lakes $T_s$ is not constant with elevation, but it can be evaluated from the storage and outflow characteristics using the differential of equation 2.26 with respect to depth ($h$).

$$T_s = \left(\frac{dS}{dh}\right)/\left(\frac{dO}{dh}\right) \tag{2.27}$$

where: $T_s$ for a given elevation ($h$) is given in units of time,
$dS/dh$ represents the slope of the storage-elevation curve at ($h$),
and $dO/dh$ represents the slope of the discharge-elevation curve at ($h$).

From equation 2.27 it can be shown that with zero inflow, the outflow recession is in the form:

$$O_2 = O_1 e^{-t/T_s} \tag{2.28}$$

where: $O_1$ = the initial outlow at time t = 1 ,
$O_2$ = the outflow at time t = 2,
and $T_s$ = the proportinality constant.

This equation is the typical decay-type function characteristic of streamflow recession, but with a varying recession coefficient for this method.

In application to natural lakes or rivers equation 2.26 may be used in the program in small finite increments of time, as shown schematically in Figure 2.1. The same general equation can be applied to channel storage by reducing the reach length to the point where "wedge" storage is negligible in comparison to "prismatic" storage. This is achieved by successive routings through many small increments of reservoir type storage, called "phases"" in SSARR terminology. The total time delay which results from in the reach channel storage can be evaluated as shown in Figure 2.2.

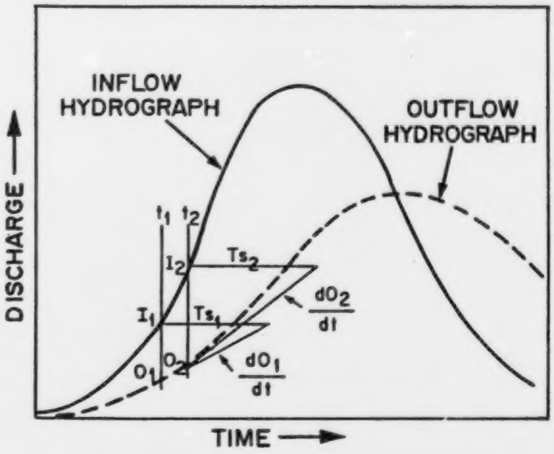

**Figure 2.1: Basic Storage Routing Procedure used in the SSARR Model**

An important feature in the routing method is that the time of storage may be varied with the magnitude of flow. The time required for flood waves to travel through a length of stream channel can be made to vary with discharge, according to channel conditions. This allows flexibility in empirically fitting relationships to observed data using constants developed from the actual record. The method may be used to closely represent the actual time delay to runoff.

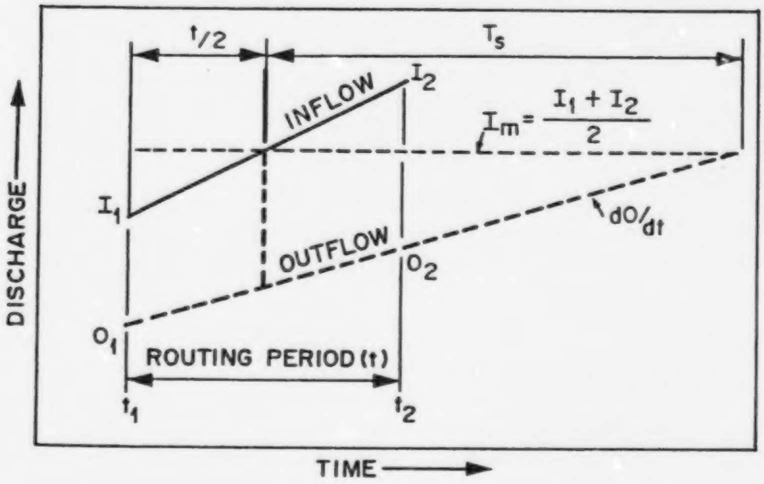

Figure 2.2: One Increment of Storage Routing

### 2.4.4 Application to a Specific Channel Reach

In this channel routing method, each channel reach is considered as representing a certain amount of storage time and the flood wave is transposed this amount by travelling through it. The total transposition is represented by the storage time $T$. The number of "phases" of reservoir storage in a particular reach determines the degree of modification of the wave form. With only one phase the modification is maximum and corresponds to the use of $X = 0$ in in the Muskingum equation. With a large number of phases, equivalent to an assumed routing time of one hour per phase, pure translation of the flood wave occurs.

The subroutine for channel routing also incorporates the ability to change the storage time as a function of discharge. The program is flexible and allows any arbitrary function of storage time with discharge. The time of storage for channel routing increments (phases) may be specified as a power function of discharge which may vary the time of storage inversely or directly. Normally, the time of storage decreases as discharge increases for inbank flows and it is convenient to express the relationship in the form of the following equation:

$$T_s = \frac{KTS}{Q^n} \tag{2.29}$$

where: $T_s$ is the time storage per increment,
$KTS$ is a constant,
$Q$ is discharge,
and $n$ is a coefficient between -1 and 1.

Routing parameter estimates for a given channel reach are usually determined in the following manner:

1. Assume an value for the exponent $n$ using examples as a guideline.
2. Determine the total time of storage for the reach (this is approximately the time of travel for the reach).
3. Estimate the number of routing phases to be used. For initial trials assume five to ten miles of river channel per phase. A larger number of phases results in less attenuation of the hydrograph peak, given a fixed total time of storage.
4. Once the number of phases is estimated, divide the total time of storage by the number of phases to obtain the time of storage per phase.
5. From the routing equation (2.29), compute the value of $KTS$ corresponding to the values of $T$, $n$, and $Q$.

Once the initial trial run is made $KTS$, $n$, and the number of phases can be adjusted to improve reconstitutions.

The discharge-time of storage relationship can also be defined through the use of a table of discharge and corresponding time of storage per phase. This allows the precise definition of relationships and can be used to simulate rather unique or unusual channel routing changes with discharge. An example of this is where the time of storage decreases with increasing discharge to bankfull stage and then increases as overbank storage begins to influence the time of storage.

## 2.5 HSPF Model

### 2.5.1 Introduction

The routing method used in the HSPF model is described by the authors (Johanson et al., 1984) as being an approximation of a kinematic wave model which uses linear reservoir concepts to achieve the routing. Kinematic wave models are generally based on the concepts proposed in a paper by Lighthill and Whitham (1955).

Kinematic wave models assume that flood wave movement is primarily kinematic in nature and that dynamic wave movement is insignificant in comparison.

The kinematic wave can be described as a monoclinal wave where there is a single value of discharge for each depth, mean flow velocity is directly related to flow depth and where these relationships can be computed using steady state flow equations such as Manning's.

Linear reservoir models can be applied to channel routing using the analogy that a stream channel is like a series of connected lakes or reservoirs. This allows the routing characteristics of the natural stream channel to be described using the simpler concepts of a level pool routing.

In a level pool routing it is assumed that storage and discharge are directly related to the depth of the lake or reservoir and therefore a single valued relationship between discharge and the volume of storage can be determined. This implies that momentum effects are insignificant in the determination of flow characteristics.

In order for this assumption to be physically valid for channel routing, where momentum effects are known to be much more important, the model must be configured in a manner to minimize dynamic wave characteristics.

The reach lengths must be short enough that wedge storage (the water contained in the dynamic disturbance) is insignificant in comparison to prism storage. The volume of storage contained within any reach for a given flow depth should be computed with the water surface parallel to the stream bed, rather than with a horizontal water surface such as is found in a lake.

As dynamic wave effects will no longer be significant within each reach, a rating curve can be computed using steady state equations which will contain a single value of discharge and volume of storage for each increment of stage.

### 2.5.2 Derivation of the Routing Equation

The river channel is conceptualized as consisting of a number of reaches, called RCHRES or reach reservoirs in HSPF terminology. The routing parameters for each reach are stated in a table called an FTABLE. This contains values of stage, surface area, storage volume and discharge. Streamflow routing only requires values of storage and discharge. The other parameters are used in auxiliary computations dealing with water quality. In general, the stream channel characteristics contained in the table are computed from channel cross-section data. The depth increment of the table depends upon the size of the cross-section, the data available, and the degree of resolution required. Intermediate values are obtained by interpolation.

The HSPF model uses the equation of continuity expressed in terms of volumes:

$$Vol_2 - Vol_1 = inflow\ volume - outflow\ volume \tag{2.30}$$

where: $Vol_2$ is the storage volume at the start of the period
$Vol_1$ is the storage volume at the end of the period

therefore:

$$Vol_2 = (inflow\ volume + Vol_1) - outflow\ volume \tag{2.31}$$

The principle task of the routing is to estimate the outflow volume and hence $Vol_2$. The outflow volume is estimated as:

$$outflow\ volume = [KS * Qout_1 + (1 - KS)\,Qout_2]\,t \qquad (2.32)$$

where: $KS$ is a weighting factor (between 0 - 0.99)
$Qout_1$ is the outflow rate at the start of the period
$Qout_2$ is the outflow rate at the end of the period

The mean rate of outflow for a given time interval is considered to be the weighted average of the rates at the start and end of the interval. The weighting factor can be used to bias the outflow rate to either the start or end of the period, in cases where trial routings indicate some adjustment of the routing parameters is required, but where there is no justification or information for changing the table entries. The weighting factor should be selected with care. The risk that the computation of outflow rates will become unstable increases as KS increases. A value of 0.5 gives the most accurate results, provided that oscillations do not occur, while the default value of 0.0 has zero risk but gives less accurate results.

Combining equations 2.31 and 2.32 yields:

$$Vol_2 = (inflow\ vol. + Vol_1) - [KS * Qout_1 + (1 - KS)\,Qout_2]\,t \qquad (2.33)$$

or simply stated, the volume of storage at the end of the period is the inflow volume plus the initial storage minus the weighted outflow volume. When the model is used for channel routing alone the rate of outflow at the end of the period is a function of the storage volume which can be expressed by:

$$Qout_2 = funct(Vol_2) \qquad (2.34)$$

or in more common symbols:

$$O_2 = funct(S_2) \qquad (2.35)$$

Routing through each RCHRES can be accomplished by the simultaneous solution of equations 2.33 and 2.34. A subroutine in the model is provided for this.

Since the HSPF model was designed for use as a comprehensive watershed model it has been provided with the facility to compute the outflow demand for a number of exits, at the end of each reach. Individual outflows can be stated as functions of storage volume, or time, or a combination of time and storage. This allows the model to simulate most possible outflows which might result from the movement of water through the natural channel, removal of water for irrigation or municipal use, or uses conditional upon supply constraints.

In the case of a simple streamflow routing the assumption is made that there is a fixed relationship between depth, surface area, storage volume and discharge, although only storage volume and discharge are used in the primary calculations. The model makes no assumptions about channel shape, but requires that the functional relationship be constant in time.

The model simultaneously solves for the discharge and storage volume at the end of the routing period ($Qout_2$ and $Vol_2$).

### 2.5.3 Determination of the Routing Parameters

Stream channel routings are normally performed for reaches where the end of reach locations are defined by gauging stations. In most of the simpler routing methods this is necessary in order to determine routing parameters such as wave speed. HSPF is a versatile routing model in that a routing configuration can be developed without any observed flows. This was necessary due to the application of the model in hydrologic modelling, where runoff from ungauged areas is simulated and routed. Since there are no observed upstream flows generally available, a routing model was developed where the routing parameters could be determined using only the physical characteristics of the stream channel.

The application of the HSPF routing subroutine requires the computation of reach surface area and storage volume for each depth increment. This is usually accomplished using surveyed cross-section data as illustrated in Figure 2.3. Discharges for the table can either be the observed discharge at a gauge location or the computed discharge for an end of reach location based upon the survey information and a steady state equation such as Manning's or Chezy's.

Due to the need to keep reach lengths short it is usually necessary to compute discharges for locations removed from gauge locations. If an observed rating table is available at a surveyed cross-section, a great deal of the uncertainty associated with the estimation of resistance coefficients can be eliminated.

It is also possible to estimate channel characteristics using topographic map information (if maps of sufficient detail are available) or from air photographs (if the technical expertise is available to interpret this information).

The weighting factor $KS$ is altered only when trial routings indicate that fine tuning is necessary.

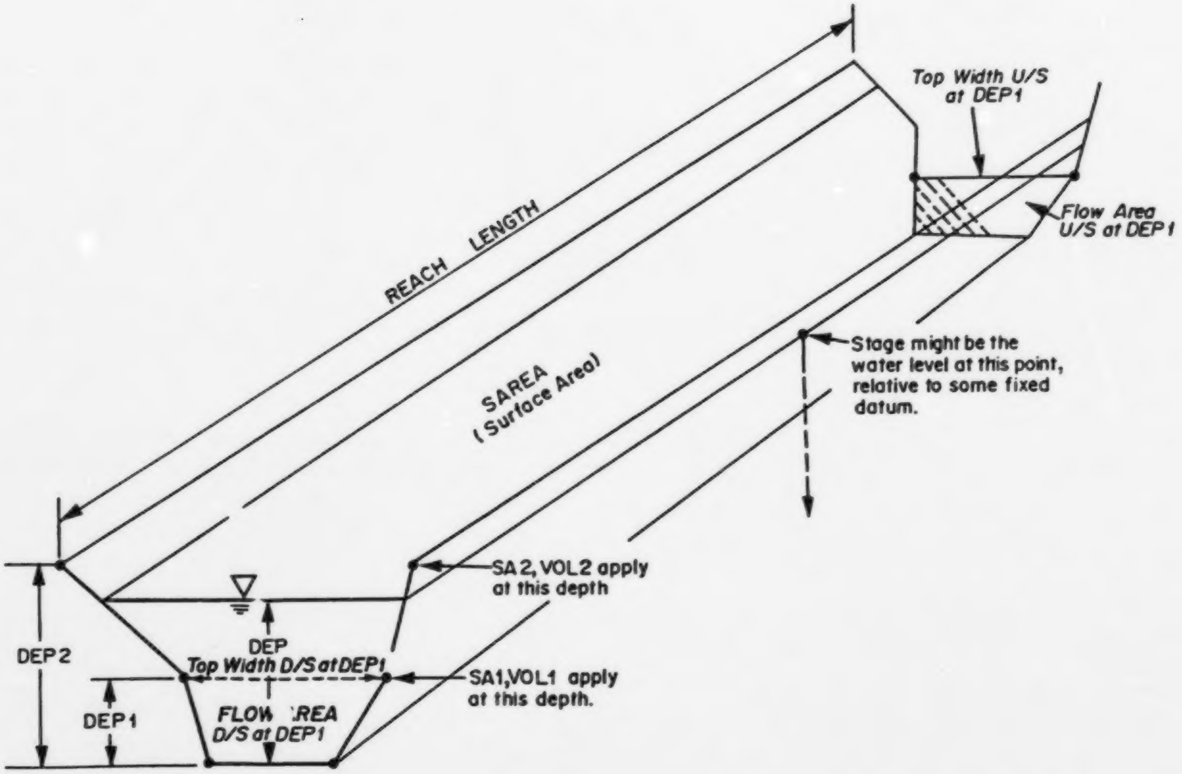
REACH LENGTH
Top Width U/S at DEP1
Flow Area U/S at DEP1
SAREA (Surface Area)
Stage might be the water level at this point, relative to some fixed datum.
SA2, VOL2 apply at this depth
DEP
DEP2
Top Width D/S at DEP1
SA1,VOL1 apply at this depth.
DEP1
FLOW AREA D/S at DEP1

**Figure 2.3: Illustration of quantities involved in calculation of FTABLES**

### 2.5.4 Application to a Specific Channel Reach

The application of the HSPF routing subroutine to a specific channel reach is straightforward, if there is sufficient cross-section data. A major advantage of this model is that wave speed is a non-linear function which can be computed using steady state flow equations and the channel-cross section information. As such, observations of the times of travel between gauge locations for various magnitudes of discharge are not required in order to set up the model.

The first step in setting up the model is to perform an inventory of cross-section and flow data available for the reach under consideration.

The reach is then subdivided into RCHRES, taking into consideration channel characteristics, the availability of data on channel characteristics, the time period of input data, and the nature of the flood hydrograph.

The end of reach should coincide with a surveyed cross-section and/or a streamflow gauge location. The parameters contained in the FTABLE are computed using the cross-section data and the length of stream channel between end of reach locations.

For each depth:

volume of storage = average area of flow * reach length
surface area = average top width * reach length
discharge = average area of flow * flow velocity computed using the Manning equation or from rating table

Trial runs will indicate whether the routing parameters are reasonably estimated. If serious errors appear to exist FTABLE discharges can be recomputed by changing the channel resistance coefficient, although this should not be necessary if streamflow measurements were available to aid in calibration. There should be relatively little error in the storage volumes if they were computed using surveyed cross-sections. Adjustment of the remaining variable, the weighting parameter (KS), should be done with caution keeping in mind its affect on computational stability and simulation accuracy.

# Bibliography

[1] Anderson, M.G. and Burt T. (editors) (1985), *Hydrological Forecasting*, John Wiley and Sons, Toronto.

[2] Chow V.T., (1959), *Open Channel Hydraulics*, McGraw-Hill Book Co., Inc., New York.

[3] Chow V.T., (1974), *Handbook of Applied Hydrology*, McGraw-Hill Book Co., Inc., New York.

[4] Cunge J.A., (1969), "On the Subject of a Flood Propogation Method", *Journal of Hydraulics Research*, IAHR, 7, pp. 205-230.

[5] Henderson, F.M., (1966), *Open Channel Flow*, McMillan, New York, New York.

[6] Hershel, Clemens, (1897), "On the Origin of the Chezy Equation", *Journal , Association of Engineering Societies*, Vol. 18, pp. 363-368. Discussion, pp. 368-369, January-June, 1897.

[7] Johanson, R.C., et al, (1984), *Hydrological Simulation Program - Fortran (HSPF) : User's Manual for Release 8.0*, EPA-600/3-84-066, Environmental Research Laboratory, Office of Research and Development, USEPA, Athens, Georgia.

[8] Lighthill M., and Whitham G.B., (1955), "On Kinematic Waves. I- Flood Movement in Long Rivers", *Proceedings, Royal Society of London*, Series A, 229 , pp. 281-316.

[9] Linsley, R.K., (1971), "A Critical Review of Currently Available Hydrologic Models for the Analysis of Urban Stormwater Runoff", *Report Hydrocomp International Inc.*, Palo Alto, California.

[10] Manning, R., (1891), "On the Flow of Water in Open Channels and Pipes", *Trans. Inst. Civil Engrs. of Ireland*, Dublin, pp. 161-207.

[11] McCarthy, G.T., (1938) "The Unit Hydrograph and Flood Routing" paper presented at the Conference of the North Atlantic Division of the Corps of Engineers, New London, Connecticut.

[12] Miller, W.A., and Cunge J.A., (1975), "Simplified Equations of Unsteady Flow" , in Mahmood, K., and Yevjevich V., (eds.) *Unsteady Flow in Open Channels*, Vol. I., Fort Collins, Colorado: Water Resources Publications.

[13] Miller, J.E., (1984), "Basic Concepts of Kinematic-Wave Models", *U. S. Geological Survey Professional Paper; 1302* United States printing Office, Washington.

[14] Meehan, P., and Wiggins, N.C., (1979), "The Last of the Muskingums", *Proceedings of the Fourth National Hydrotechnical Conference - River Basin Management*, CSCE, Vol. 1, pp. 332-345.

[15] Natural Environment Research Council, (1975), *The Flood Studies Report, Volume III, Flood Routing Studies*, Whitefriars Press, London.

[16] Ponce, V.M., and Yevjevich V., (1978), "Muskingum-Cunge Method With Variable Parameters", *Journal of the Hydraulic Division*, ASCE, 104, (HY12), pp. 1663-1667.

[17] Ponce, V.M., Li, R.M., and Simons, D.B., (1978), "Applicability of Kinematic and Diffusion Models", *Journal Hydraulic Division*, ASCE, 104 (HY3), pp. 353-360.

[18] Puls, L.G., (1928), "Construction of Flood Routing Curves", *House Document 185*, US 70th Congress, 1st session, Washington, DC, pp. 46-52.

[19] Rockwood, D.M., (1958), "Columbia Basin Stream Flow Routing Computer", *Transactions*, ASCE, 126(4), pp. 32-56.

[20] Saint-Venant A.C. Barre de, (1871), "Theorie du mouvemnt non permanent des eaux, avec application aux crues des rivieres et a l'introduction des marees dans leur lits", (Theory of the nonpermanent movement of waters with application to the floods of rivers and to the introduction of the tides within their beds), *Comptes rendus des seances de l'Academie des Sciences*, vol. 73, pp. 147-154 and 237-240.

[21] Tatum, F.E., (1940), *An Simplified Method of Routing Flood Flows Through Natural Valley Storage*, (unpublished memorandum), U.S. Engineer's Office, Rock Island, Ill..

[22] Viessman, W., Knapp, J.W., Lewis, and Harbaugh, T.E., (1977), *Introduction to Hydrology*, Dun-Donnely Publishers, New York, New York.

[23] Weinmann, P.E., and Laurenson, E.M., (1979) "Approximate Flood Routing Methods: a Review", *Journal Hydraulic Division*, ASCE, 105 (HY13), Proc., Paper 15057.